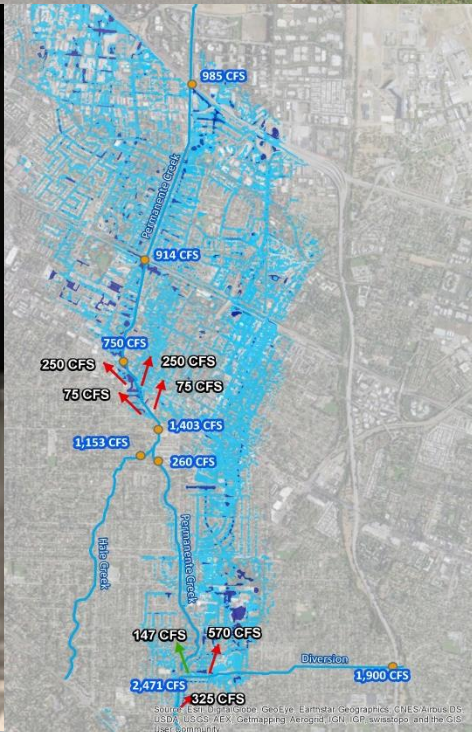
Santa Clara Valley Water District

Permanente Creek Flood Protection Project

CONFORMED HYDRAULIC DESIGN REPORT

Prepared for
Santa Clara Valley Water District

August 31, 2016



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Schaaf & Wheeler
CONSULTING CIVIL ENGINEERS
1171 Homestead Road, Suite 255
Santa Clara, CA 95050
(408) 246-4848

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The following report has been prepared under the supervision of the undersigned, who certifies that he is a Registered Civil Engineer in the State of California.

Prepared by:

CHARLES D. ANDERSON
No. 43776

8/31/16
CHARLES D. ANDERSON, RCE No. 43776 (Date)

Schaaf & Wheeler
CONSULTING CIVIL ENGINEERS
1171 Homestead Road, Suite 255
Santa Clara, CA 95050
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INTRODUCTION

This conformed basis of hydraulic design has been prepared for the Santa Clara Valley Water District (District) to document hydraulic analyses of Permanente Creek and the Permanente Diversion Channel for the Permanente Creek Flood Protection Project (Project). The Project is part of the voter approved Safe Clean Water Program. The objectives for the project are to provide flood protection to homes downstream (north) of El Camino Real, prevent the flooding of Middlefield Road and Central Expressway, develop an asset protection plan for the deteriorating concrete channels built previously, and develop long term maintenance guidelines for the creek, while minimizing the cost of maintenance.

This report describes the hydraulic performance of Permanente Creek between San Francisco Bay and Rancho San Antonio County Park in Los Altos, and the Permanente Diversion Channel to Stevens Creek in Mountain View. The hydraulic designs for two new storage facilities at Rancho San Antonio County Park and McKelvey Park including inlet and outlet structures, emergency spills, and basin drainage facilities at each location are also described. Hydraulic and engineering analyses for Permanente Creek described herein are based upon the District's 90% Plan Set dated March 2014. Hydraulic models have also been conformed to final plans for Rancho San Antonio County Park storage area prepared by Mott MacDonald (March 17, 2014) and final plans for the McKelvey Park storage area also prepared by Mott MacDonald (December 7, 2015).

Project Objectives

The basic objective of the Permanente Creek Flood Protection Project is to protect properties along Permanente Creek downstream of El Camino Real against the base flood in a manner consistent with the policies and design criteria of the District and the Federal Emergency Management Agency (FEMA) as administered through the National Flood Insurance Program (NFIP). This basic objective will be accomplished in part through the use of off-stream flood detention facilities at Rancho San Antonio County Park near Los Altos and McKelvey Park in Mountain View.

District and NFIP regulations define the "base flood" as a flood having a one percent chance of being equaled or exceeded in any given year. Often this is referred to as a "one-percent" or "100-year" flood. This should not be confused with a flood that *will* occur once every one hundred years, but one that might occur once every one hundred years *on the average* over a very long period of time.

Over the life of a 30-year mortgage, there is a 26 percent chance of experiencing a flood equal or greater in magnitude than the base flood as demonstrated by Table 1. These probability concepts may have an impact on construction staging, especially the initiation and completion of work within particular reaches, and the timing of floodwall construction.

Table 1
Relative Risk of Various Flood Events

	5-year	10-year	100-year
Annual risk of event	20%	10%	1%
Risk of at least one event in 5 years	67%	41%	5%
Risk of at least one event in 10 years	89%	65%	10%
Risk of at least one event in 30 years	99.88%	96%	26%
Risk of at least one event in 50 years	99.998%	99%	39%
Risk of at least one event in 100 years	really high	99.997%	63%

Although the primary project objective is to provide a level of flood protection along Permanente Creek meeting District and FEMA standards, additional project objectives include:

- Balancing flood reduction with environmental restoration, community beautification, and recreation opportunities.
- Providing efficient and cost-effective flood control maintenance to ensure that the District is able to maintain design system performance in the future.
- Engendering community support for the proposed project.

Scope of Hydrologic and Hydraulic Design

District-supplied hydraulic models for Permanente Creek and the Permanente Diversion Channel for both existing and design conditions have been updated based on the now FEMA-approved HEC-RAS Version 5.0, record drawing reviews, and field reconnaissance. Unsteady backwater and forewater analyses have been completed to evaluate existing and design capacities within the project reaches. District-supplied hydrologic models have been used to route flood hydrographs through proposed flood protection facilities, thus enabling a comparison of flow to available capacities within project reaches. HEC-RAS models have also been constructed to model the performance of proposed flood storage basins and their accompanying hydraulic structures.

District project reaches include Permanente Creek from San Francisco Bay to the Rancho San Antonio County Park; Hale Creek from the Hale-Permanente Confluence to Foothill Expressway; and the Permanente Diversion Channel to Stevens Creek. Each reach is depicted in Figure 1. Permanente Creek Reach P1 extends from San Francisco Bay upstream to the Boat Pond Bridge at Project Station 62+76; Reach P2 encompasses the improvement project from Boat Pond Bridge upstream to US Highway 101 at Project Station 131+79; Reach P3 extends from Highway 101 upstream to Villa Street at Project Station 200+32; Reach P4 extends from Villa Street to upstream of El Camino Real at Project Station 237+20; Reach P5 extends from upstream of El Camino Real to the Hale-Permanente Confluence near Raymundo Avenue at Project Station 262+91; Reach 6 extends from the confluence upstream to Cuesta Drive at Project Station 302+86; Reach P7 extends from Cuesta Drive upstream to the Permanente Diversion structure at Station 345+24; and Reach P8 extends from the Permanente Diversion structure upstream to Foothill Expressway at Station 395+23. In addition to the original Reaches P1 through P8 as defined by the District, a ninth reach, Reach P9, which extends from Foothill Expressway upstream to Rancho San Antonio County Park at Station 506+32 is defined to model the proposed facility at the park. The Permanente Diversion Channel is known as Reach PD. Hale Creek consists of two reaches: Reach H1 from the confluence to Cuesta Drive at Station 4356.95 and H2 from Cuesta Drive to Station 9481.62 at Foothill Expressway.

Sources of Data Used in Analyses

The primary sources of geometric data used in this evaluation have been obtained from the District, with some supplemental field surveys performed by Mott MacDonald. Design discharge hydrographs are as provided directly by the District. Available record drawings have been used to verify some of the data as appropriate.

Report Outline

After briefly summarizing the facilities outlined in the District's 2008 Permanente Creek Flood Protection Project Planning Study Report, the final project configuration is discussed and routed design discharges are presented for a comparison to updated channel capacities. Each storage basin's final design is described with an evaluation of its effectiveness in attenuating the one-percent flood hydrograph. A post-project one-percent floodplain is compared to the existing conditions floodplain and project objectives. Post-project ten-percent hydraulic performance is also evaluated.

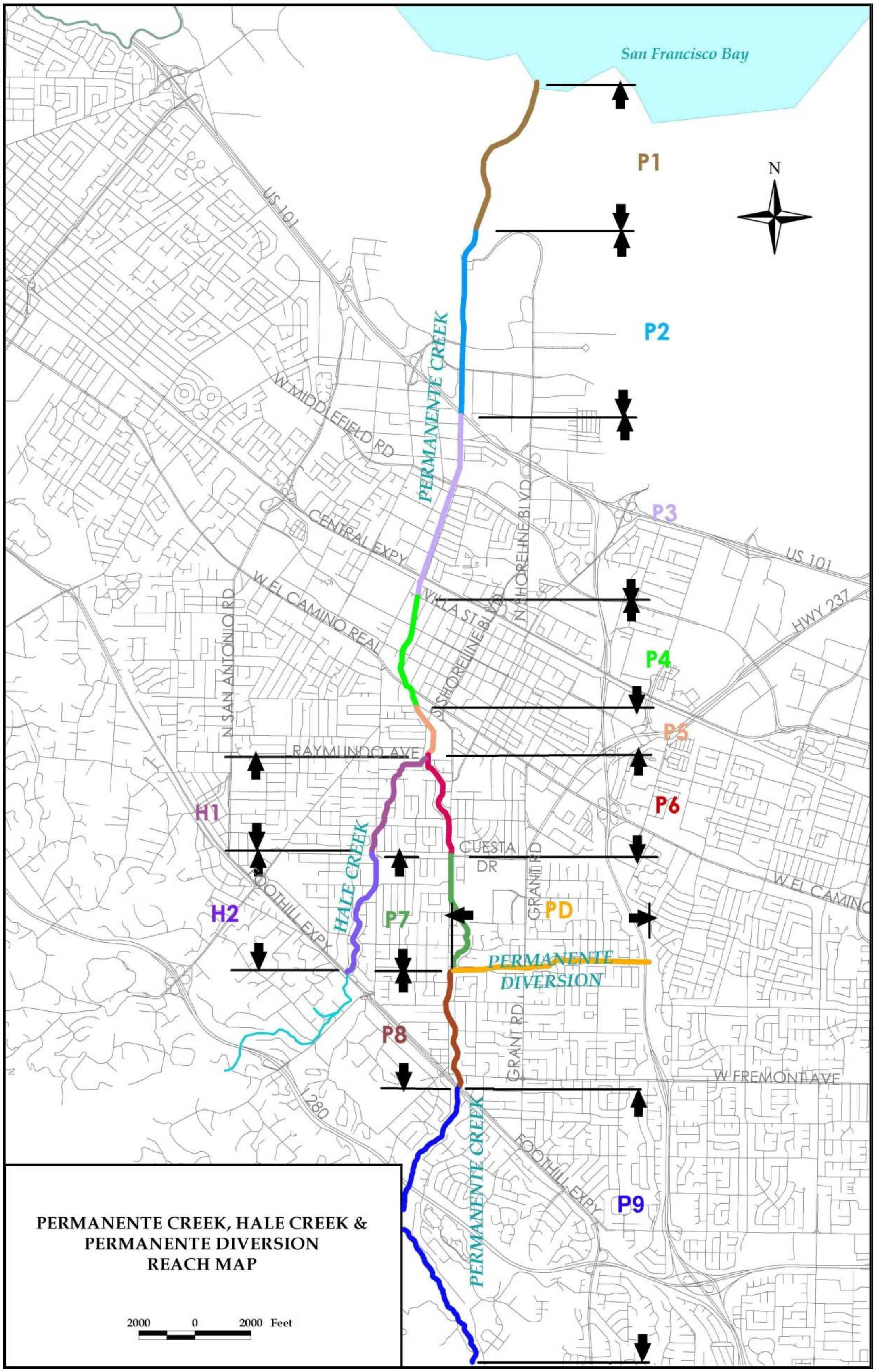
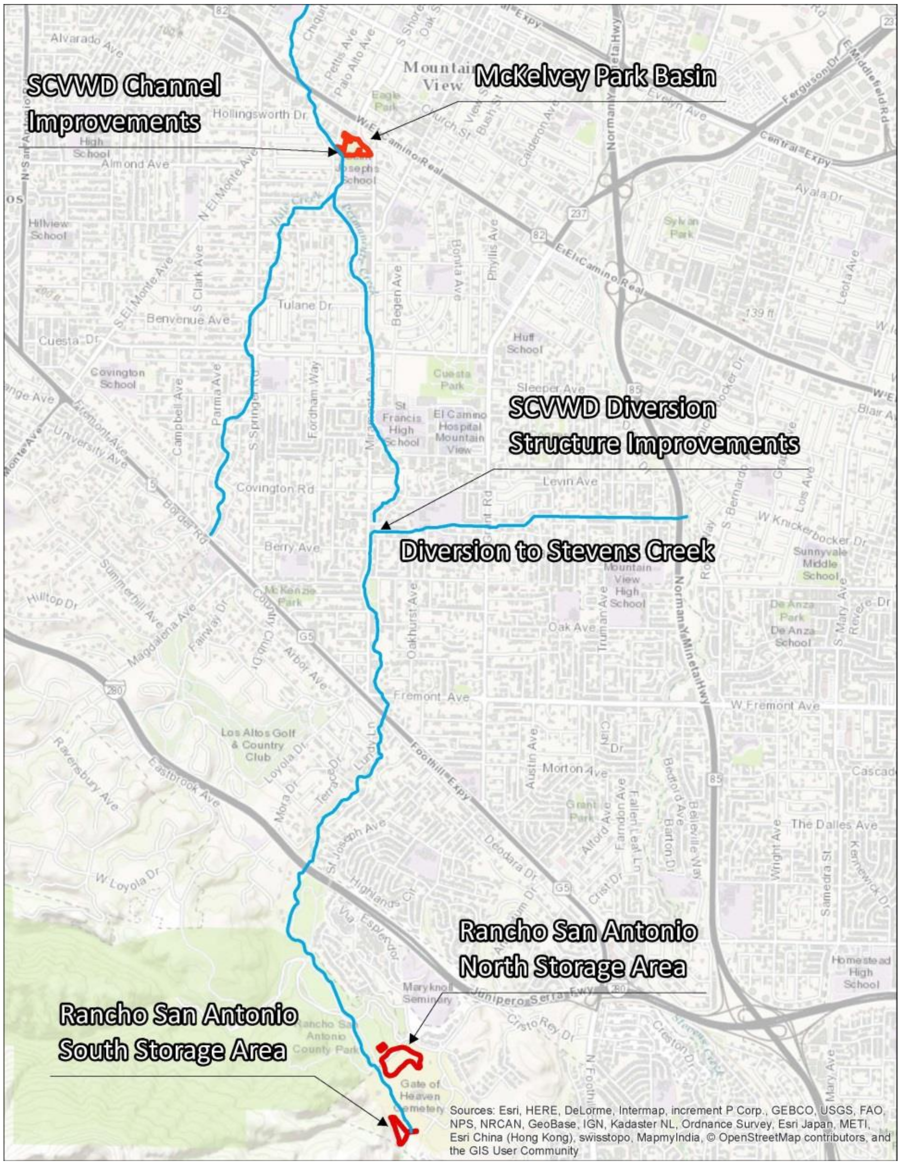


Figure 1: Permanente Creek Reach Designations

PROJECT ELEMENTS

In July 2008 the District completed the "Permanente Creek Flood Protection Project Planning Study". The planning study recommended off-stream flood detention facilities at Rancho San Antonio County Park, Blach Intermediate School, Cuesta Park Annex and McKelvey Park, a bypass channel along Hale Creek, channel widening along selected reaches of Permanente Creek and Hale Creek, floodwalls atop levees along Permanente Creek north of U.S. Highway 101, and a new diversion structure at the Permanente Diversion. The final Project configuration is shown in Figure 2. During design development, the Rancho San Antonio County Park detention facility was divided into two separate basins based on detailed topography not available previously, the Blach Intermediate School basin was eliminated, and the Cuesta Park Annex basin was eliminated.


SCVWD Channel Improvements
McKelvey Park Basin
SCVWD Diversion Structure Improvements
Diversion to Stevens Creek
Rancho San Antonio North Storage Area
Rancho San Antonio South Storage Area

Sources: Esri, HERE, DeLorme, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), swisstopo, MapmyIndia, © OpenStreetMap contributors, and the GIS User Community

Figure 2: Project Elements

PROJECT DESIGN

The Permanente Creek Flood Protection Project stores flood flows that exceed the conveyance capacities of Permanente Creek and the Permanente Diversion as needed to prevent one-percent flooding north of El Camino Real. This report documents conveyance capacities and the design of storage facilities to meet the project's hydraulic performance objective. In essence this task compares estimated flood flows in Permanente Creek and the Permanente Creek Diversion after the storage basins are constructed, to estimated channel capacities with District-planned system improvements, based on the engineering documents referenced above.

Design Discharges

Project design base flood discharges summarized in Table 2 and shown on Figure 3 are those provided by the District in October 2011. It should be emphasized that the discharges provided in Table 2 do not necessarily reflect channel capacity limitations. Reported discharges are based on the assumption that all water would be contained in or return immediately to the creek without overland flow away from the generally perched channels.

Table 2
Permanente Creek Design Discharges

Location	**Drainage Area (mi^2)**	**Peak Discharge (cfs)**	
		10-year Q10%	**100-year Q1%**
West Branch Permanente Creek	3.51	452	908
Upper Permanente Creek u/s West Branch Permanente Creek (P1A)	1.17	257	450
Lower Permanente Creek u/s West Branch Permanente Creek	2.48	578	957
Permanente Creek u/s West Branch Permanente Creek (P1)	3.65	757	1,350
Permanente Creek d/s West Branch Permanente Creek	7.16	1,186	2,232
Permanente Creek @ Diversion (P2)	8.05	1,303	2,416
Permanente Diversion to Stevens Creek	8.05	1,204	2,316
Permanente Creek u/s Hale Creek	9.23	208	209
Hale Creek @ Fremont S.C. #33 (P3)	2.70	501	847
Hale Creek u/s Permanente Creek	4.39	715	1,154
Permanente Creek d/s Hale Creek (P4)	13.62	922	1,363
Permanente Creek @ Central Expy (P5)	15.40	1,081	1,525
Permanente Creek @ US 101 (P6)	16.17	1,147	1,594

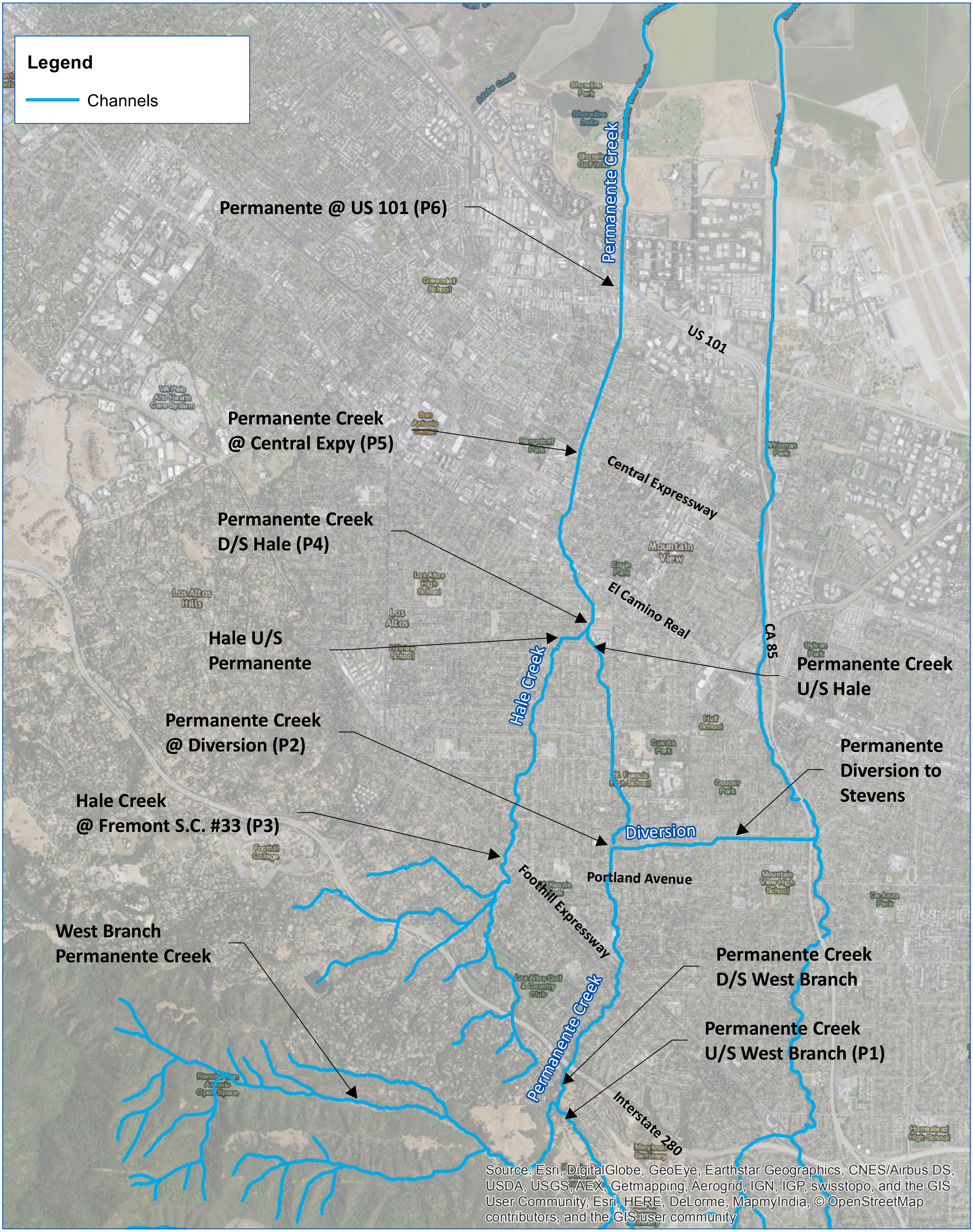


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FIGURE 3: **Permanente Creek Design Discharge Locations**

Hydraulic Analyses

Preliminary hydraulic models of Permanente and Hale Creek were provided by the District and developed into the final conformed design hydraulic models as part of this work. The project model is divided into a series of three separate steady state and unsteady hydraulic models for system analysis as depicted in Figure 4Figure . Both of the proposed detention facilities are fully incorporated into the final unsteady hydraulic models using HEC-RAS 5.0, with an overland two-dimensional flow grid established to trace overflows under both existing and post-project conditions.

The upstream unsteady model (“Model 1”) consists of Permanente Creek from a point upstream of Rancho San Antonio County Park to the Permanente Diversion Channel and the Permanente Diversion Channel to Stevens Creek (reaches P8, P9 and PD). A two-dimensional overflow grid is attached to the reach between the County park and Stevens Creek. The Permanente Diversion is modeled as a lateral outflow structure with non-diverted low flows exiting the model as a flow hydrograph. The unsteady downstream model (“Model 2”) consists of Permanente Creek from the Permanente Diversion to El Camino Real. The steady-state downstream model (“Model 3”) picks up at El Camino Real and continues to San Francisco Bay. This model is run in the steady-state mode to check channel capacity at peak discharge and avoid stability issues associated with the long underground culvert beginning at El Camino Real.

Boundary Conditions

The upstream model boundary condition consists of the inflow hydrograph derived from the District’s HEC-HMS analyses for Permanente Creek above Rancho San Antonio County Park. Stevens Creek forms the downstream boundary to “Model 1”. An HEC-RAS model of Stevens Creek developed as part of the levee accreditation process in 2009 has been used to establish a stage hydrograph at the confluence with the Permanente Diversion Channel, using hydrographs from the District’s combined Stevens Creek and Permanente Creek watershed HEC-HMS model. For the post project plan in “Model 1,” the upstream boundary is derived from the downstream flow hydrograph from the Rancho San Antonio Unsteady HEC-RAS model. The Rancho San Antonio HEC-RAS model includes all project components for the Rancho San Antonio County Park Flood Protection Improvements. The Hale Creek flow hydrograph is input as a lateral inflow to “Model 2.” The downstream boundary condition for “Model 3” is the higher-high coincident tidal elevation at the creek mouth as described later in this report. “Model 1” is linked to “Model 2” by the Permanente Diversion flow hydrograph, which serves as the upstream boundary condition for “Model 2”. “Model 3” has been used with the flow hydrograph at the downstream boundary of “Model 2” to establish a stage hydrograph where “Model 2” is linked to “Model 3” at El Camino Real.

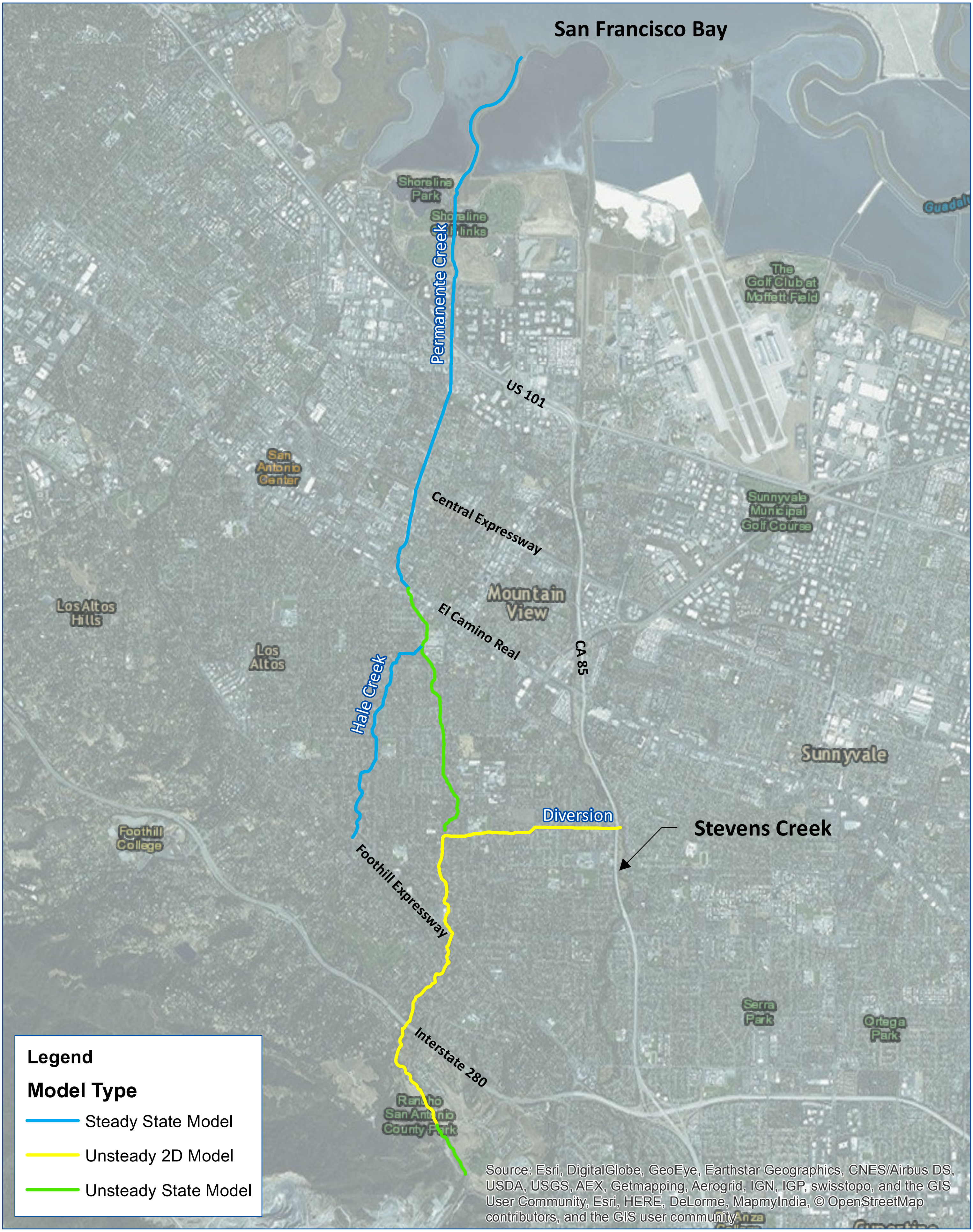

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FIGURE 4: **Hydraulic Model Schematic**

HYDRAULIC METHODOLOGIES

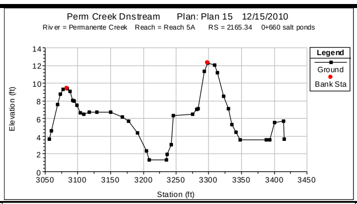
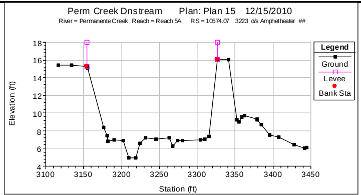
To estimate design reach capacities and to provide design parameters used for the hydraulic structures at the off-stream storage facilities, HEC-RAS models have been prepared to reflect project improvements as described in the 90% design level plans for Permanente Creek prepared by the District, and final plans for Rancho San Antonio County Park and McKelvey Park prepared by Mott MacDonald. Relevant hydraulic modeling parameters including channel roughness, bridge modeling, transition losses and boundary conditions are described herein.

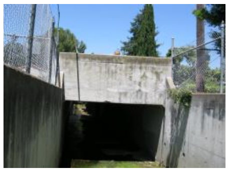
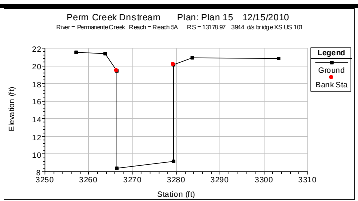
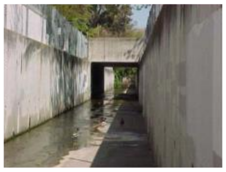
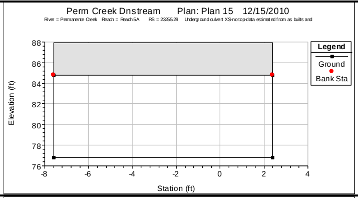
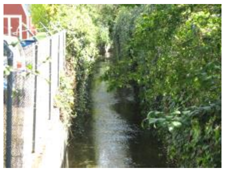
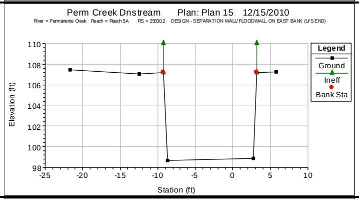
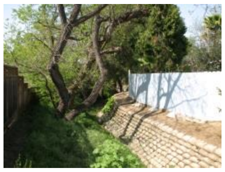
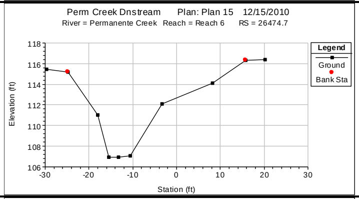
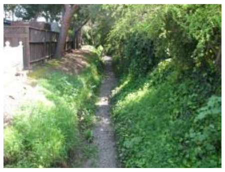
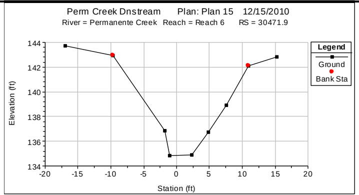
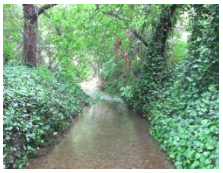
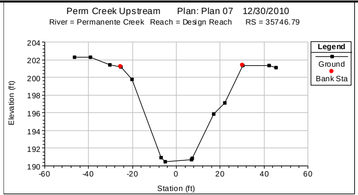
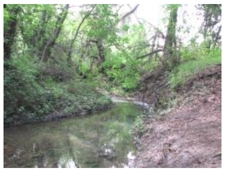
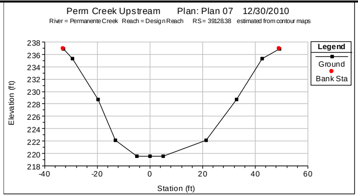
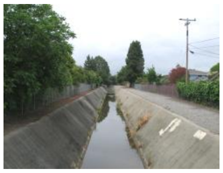
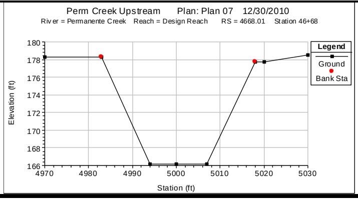
Channel Roughness

In one-dimensional open channel flow analysis, "Manning's n" is used to represent the retarding forces to flow imposed by the channel bed and banks. Roughness elements along the wetted perimeter of Permanente Creek will vary across an individual cross section. For instance, the channel cross section might eventually contain elements of grassed banks, mature riparian vegetation including trees, shrubs or brush, access roads, emergent wetland areas and/or open water, bare earth and concrete walls.

To compute water surface elevations in a channel with variable roughness using a one-dimensional model (i.e. with a mean velocity), it is necessary to estimate an effective (composite) roughness value for each cross section. Table 3 lists estimates of composite roughness used in the channel capacity analysis and illustrates typical cross sections and vegetation representing each project reach of Permanente Creek. Composite roughness coefficients are estimated based on field reconnaissance with environmental conditions projected to winter flood flows, review of literature regarding Manning's "n" values, and previous experience with flood channel and floodplain analyses.

Table 3
Channel Roughness Coefficients Used for Permanente Creek Analysis

Reach	Vegetation	Representative Section	Composite Roughness (Manning's n)
P1			.030
P2			.030

Reach	Vegetation	Representative Section	Composite Roughness (Manning's n)
P3			.015
P4			.015
P5			.015
P6			.035
P7			.040
P8			.045
P9			.050
PD			.015

Transition Losses

An energy loss takes place just upstream and downstream from each structure as flow contracts and expands into and out of a bridge, culvert, or other channel transition. The following contraction and expansion coefficients for channel transitions are used in the hydraulic model (Table 4).

Wherever turbulent conditions create the potential for energy loss, contraction and expansion coefficients are increased. Other transitions include channel bends, resting pools, and maintenance access ramps. For gradual transitions or no transitions between channel sections, the default contraction coefficient (C_i) is 0.1 and the default expansion coefficient (C_o) is 0.3.

Table 4
Contraction and Expansion Coefficients

Transition Type	Contraction	Expansion	Source
Gradual	0.1	0.3	HEC
Warped	0.1	0.2	Chow
Wedge	0.3	0.5	Chow
Square End	0.3	0.75	Chow
Abrupt	0.6	0.80	HEC

Bridge and Culvert Modeling

In addition to the energy required to overcome channel resistance, structures such as bridges and culverts also cause energy losses, which can result in a raised water surface profile. Several methods are available through HEC-RAS to compute energy losses through a bridge. The "energy only" or standard step method handles a bridge section without piers in the same manner as a natural river section, except that the area between the low chord of the bridge (soffit) and the top of road is subtracted from the total cross-sectional area, and the wetted perimeter is increased where water is in contact with the bridge. Increased frictional resistance due to the added wetted perimeter is included in the energy loss through the structure.

When the computed water surface elevation is above the bridge soffit, a "pressure/weir" feature can compute losses through the structure for pressure (orifice) flow, weir flow over the top, or a combination of both. Changes to the water surface profile resulting from the bridge are calculated based on hydraulic formulae that estimate the change in energy and water surface elevation through the bridge. FHWA Chart 8 is used to model the box culvert sections. The energy loss coefficient at the culvert entrance is set to 0.5 reflecting sharp edged culvert entrances with no significant rounding.

Hydraulic Stability

Flood flows can be carried within channels either as subcritical flow or supercritical flow, often referred to as "tranquil" and "rapid" flow, respectively. The flow regime is defined by how fast the water is moving relative to the velocity of the wave that results from a small disturbance in the water surface. Disturbances in subcritical flow move upstream; disturbances in supercritical flow cannot move upstream because such waves must be swept downstream. The Froude number (F_r), which is analogous to the Mach number for gas flow, is defined as the ratio of stream velocity to wave velocity:

$$F_r = \frac{v}{\sqrt{g\,y}}$$

where v = stream velocity (feet per second)
g = gravitational acceleration (feet per second squared)
y = water depth (feet)

A Froude number greater than one signifies supercritical flow (stream velocity greater than wave velocity), while a Froude number less than one indicates subcritical flow. When the Froude number is between 0.8 and 1.2, however, the flow can be unstable, characterized by standing waves and other disturbances that may tend to propagate upstream or downstream depending upon the state of flow. Critical flow is assumed as the upstream boundary condition of each model as well as the downstream boundary condition of the Permanente Diversion.

Tidal Boundary Condition

A coincident one-percent tide analysis has been performed to establish the downstream boundary for Permanente Creek at San Francisco Bay. The 19-year mean tide cycle for San Francisco Bay represents average tide heights over a specific period known as the tidal epoch, which spans the 19 years it takes for every possible combination of relative positions for the sun, moon and earth to occur. This cycle consists of two high tides (one higher than the other) and two low tides (one lower than the other) each lunar day.

Based on calculations for relative celestial positions, it is possible to predict tides for any day of the year at any time of the day. Astronomic tides, created by the gravitational forces of the moon and sun acting on earth's oceans, are provided in tide prediction calendars. The mean tide cycle is simply the long-term average of astronomic tides. Observed tides, on the other hand, are actual tidal elevations recorded by National Oceanic and Atmospheric Administration (NOAA) gaging stations located throughout coastal areas.

Traditionally, Mean Higher High Water (MHHW) has been used as the backwater condition where riverine (freshwater) runoff meets an estuarine (saltwater) body. However, evidence shows that mean tide elevations are not necessarily an appropriate boundary condition during storm events and tide elevations in San Francisco Bay are elevated (relative to predicted tides) during periods of heavy rainfall. Furthermore, the relationship between coincident tides and maximum annual runoff can be quantified and used in the model, providing for a more statistically correct solution than an arbitrarily selected tide condition.

The El Niño storm of February 2-3, 1998 provided an ideal event for examining potential correlations between runoff events and tide action. While stream runoff as measured by local gages often approached historic recorded levels, observed tides in San Francisco Bay were substantially higher than predicted.

Figure 5 shows predicted and recorded tides in early February 1998 at Redwood City, which has the closest NOAA tide gage to Permanente Creek. Recorded tides during the week of this runoff event were consistently higher (on the order of up to 4 feet) than the astronomic (predicted) tide heights due to storm surge. As a control, observed tide heights are compared to predicted tides six months later at the same station, using the same sets of data. Figure 6 shows tide elevations during early August 1998, when there is very close agreement between the predicted and the actual tides and no rainfall. Both figures present tides on the local Mean Lower Low Water (MLLW) datum.

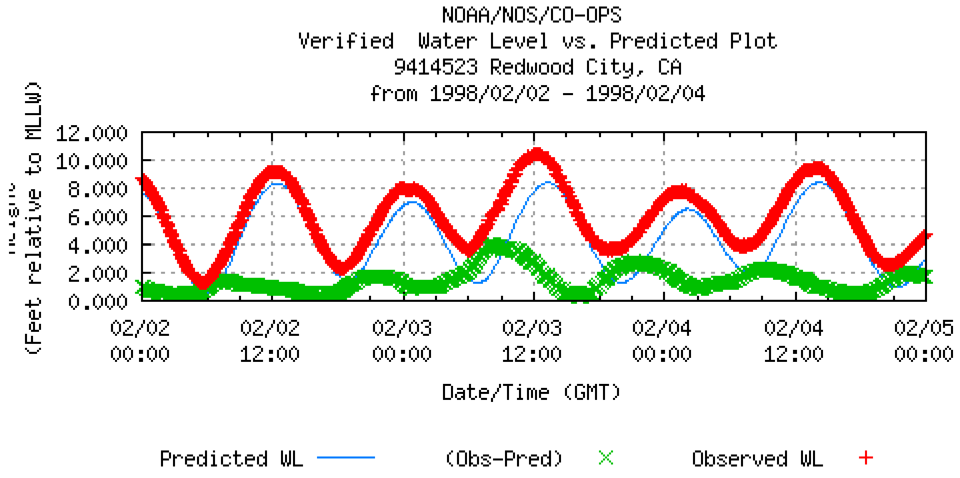


Figure 5: San Francisco Bay Tide During Feb. 2-3, 1998 Storm Event

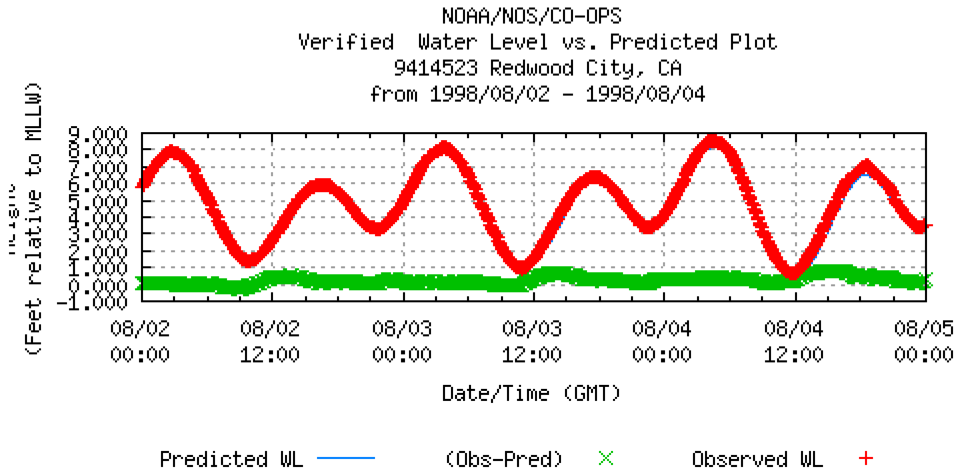


Figure 6: San Francisco Bay Tide During Early August 1998

Historic tide records have been examined to see whether the phenomenon demonstrated in February 1998 at Redwood City occurred elsewhere in the Bay Area and during other heavy runoff events in the past. Results of this investigation presented in Table 5 indicate that during the 1998 runoff event, similar rises in tide elevations (over astronomic) were experienced at other recording tide stations in the Bay.

Table 5
Tides During February 1998 Storm Surge

Location	Maximum Difference Between Predicted and Recorded Tides in feet	
	Higher High	Lower Low
Golden Gate	2.0	2.9
Alameda	2.0	2.7
Redwood City	2.0	2.7
Monterey Harbor	1.7	1.8

The observed phenomenon presented in Table 5 is not strongly dependent upon tide gage location, particularly within San Francisco Bay, and is exhibited during many historic storm events. Data indicate that higher tides as observed during the February 1998 event are not an isolated incident; rather, higher than predicted tides can be expected during storm events that generate significant runoff. Increases in the data set between observed tides over predicted tides range from 0.3 foot to 2 feet for the highest tide, and from 0.9 foot to 3 feet for the lowest tide.

From observed historical data, it appears that storm-related forces induce higher tides during rainfall events, and by extension, runoff events. This phenomenon may be due to a number of meteorological or hydrologic factors. NOAA refers to the term "inverse barometer effect", and defines it as higher tides that are caused by lower barometric pressures associated with winter storm systems. References to "storm surges", the meteorological effects of low barometric pressures and/or strong southerly winds, are also found in the literature. The exact nature and cause of this phenomenon, however, are not as important as potential impacts to backwater conditions for Stevens Creek. To model an appropriate San Francisco Bay tidal cycle during the one percent storm event, elevations for each critical point in the tide cycle are adjusted based on the one-percent conditional probability of coincident occurrence with the annual maximum discharge of San Francisquito Creek at Stanford, which represents the closest USGS stream flow gauging location with sufficient length of record for analysis.

This procedure is as described by Dixon (1986), whose hypothesis was that high tide events tend to occur the same day as flood flow events using conditional probability:

$$P(x,y) = P(x|y)\ P(y)$$

where $P(x,y)$ is the probability of occurrence of x and y; $P(x|y)$ is the probability of occurrence of x given y; $P(y)$ is the probability of occurrence of y; x is tide elevation; and y is maximum annual peak discharge. Since we are interested only in annual maximum discharges, $P(y)$ is one and the probability of joint occurrence, $P(x,y)$, is equal to the probability of x given y.

Coincident higher high tides are fitted to a probability curve using the median plotting position for every recorded tide extreme at San Francisco (Presidio/Golden Gate) that occurred within 24 hours of the recorded maximum annual discharge. Figure shows the probability distribution on the MLLW datum.

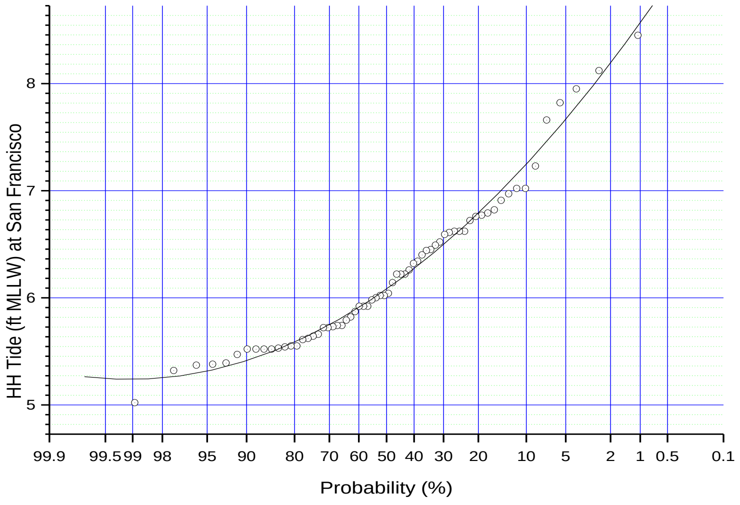


Figure 7: Coincident 1% Tide at San Francisco

The coincident one-percent high tide elevation at San Francisco is 8.63 feet MLLW. A tide elevation at the Golden Gate can be corrected for a different location within San Francisco Bay. The correction at the mouth of Permanente Creek is to add 2.6 feet to high tides. On the MLLW datum, the coincident one-percent tide for Permanente Creek is 11.23 feet. By subtracting 4.1 feet, the tide is converted to 7.13 NGVD (USACE, 1984). Using the Vertcon software, 2.68 feet are added at this location to produce a coincident one-percent high tide at the mouth of Permanente Creek of ***9.8 feet NAVD***.

The timing of coincident tide elevations with peak rainfall/runoff is also a random process. Since there are not sufficient data to statistically analyze the impact of tide timing, a constant high tide is assumed for the duration of the discharge hydrograph. Since the project reach is upstream of El Camino Real, which itself is 4.4 miles upstream of the tidal boundary condition, hydraulic conditions within the project reach are not affected by this conservative assumption.

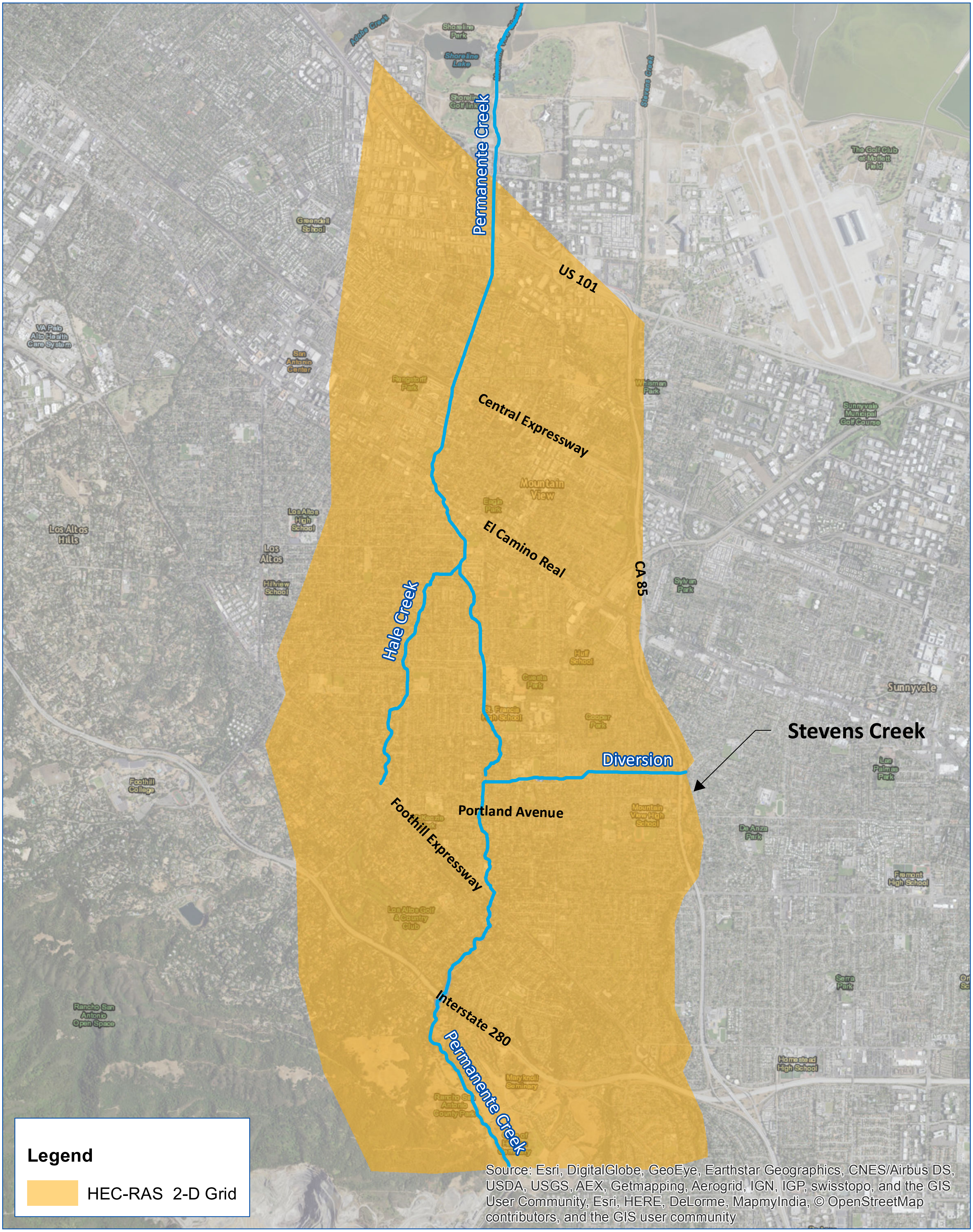
Overland Flow Routing

HEC-RAS is used to establish spill hydrographs for 100-year discharge that exceeds bank-full channel capacity given the lateral structure weir coefficients for levees, floodwalls and high ground at channel banks described by Table 6, which is based on HEC recommendations.

Table 6. Weir Coefficients used for Lateral Structures

Modeled Lateral Structure Type	Description	Range of Weir Coefficients	Coefficient Used
Concrete Floodwall / Levee 3 feet higher or more above natural ground	Broad crested weir shape. Flow over levee acts like weir flow.	2.6 – 3.2	2.6 / 3.0
Levee 3 feet higher or more above natural ground.	Broad crested weir shape. Flow over levee acts like weir flow.	1.5 – 2.2	2.0
Levee 1 to 3 feet elevated above ground.	Broad crested weir shape. Flow over levee acts like weir flow but submerged easily.	1.0 – 2.0	1.5
Natural high ground barrier 1 to 3 feet high.	Does not really act like a weir, but must flow over high ground to get into 2D or storage area.	0.5 – 1.0	1
Non-elevated overbank terrain.	Overland flow escaping the main channel.	0.1 – 0.5	0.3

The depth of flow within overbank floodplains is calculated within a grid of variable-dimension cells using the HEC-RAS program. The extent of this grid is shown as Figure 8. Overbank flow modeling is based on ground information from the 2006 Santa Clara County LiDAR point cloud. The geographic extents of the grid have been extended iteratively as necessary to ensure that all wetted areas are captured based on the two-dimensional flow analysis. That is, the grid is large enough to include dry cells along the entirety of its perimeter for all modeled overflow scenarios.



Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community, Esri, HERE, DeLorme, MapmyIndia, © OpenStreetMap contributors, and the GIS user community

FIGURE 8: **Grid for 2D Overbank Analysis**

Overbank Flow Roughness Coefficients

HEC-RAS 5.0 now supports the use of land cover datasets to estimate the spatial variability in surface roughness for shallow flooding areas. The National Land Cover Database (2011) is utilized for the project study area and represents the most recent land cover shapefile produced by the Multi-Resolution Land Characteristic Consortium. The NLCD layer utilizes a 16-class system for identifying land covers similar to the previously developed NLCD layers. The NLCD shapefile is imported into the HEC-RAS model and converted to a raster grid for association to the various 2-D grid elements. A lookup table of Manning's overland flow roughness coefficients (n values) is used to assign a unique roughness for each land cover classification. The lookup table used in the model to assign overland flow coefficients based on land cover is given as Table 7.

Table 7. Roughness Coefficients for Overland Flow

Land Cover Type	Manning's n Value
Barren Land Rock/Sand/Clay	0.030
Developed, high intensity	0.100
Developed, low intensity	0.050
Developed, medium intensity	0.075
Developed, open space	0.013
Emergent Herbaceous Wetlands	0.100
Evergreen Forest	0.120
Grasslands/Herbaceous	0.030
Mixed Forest	0.120
Open Water	0.030
Shrub/Scrub	0.050
Woody Wetlands	0.100

ANALYSIS OF EXISTING FLOOD CONDITIONS

HEC-RAS 5.0 is used in its two-dimensional unsteady-state mode to develop spill hydrographs at locations where the estimated one-percent flow exceeds channel capacity under existing conditions without the completion of hydraulic structures on Permanente Creek and storage basins at the Rancho San Antonio County Park and McKelvey Park. Spill hydrographs are routed through channel overbank areas, producing a mapped floodplain to which post-project performance may be compared.

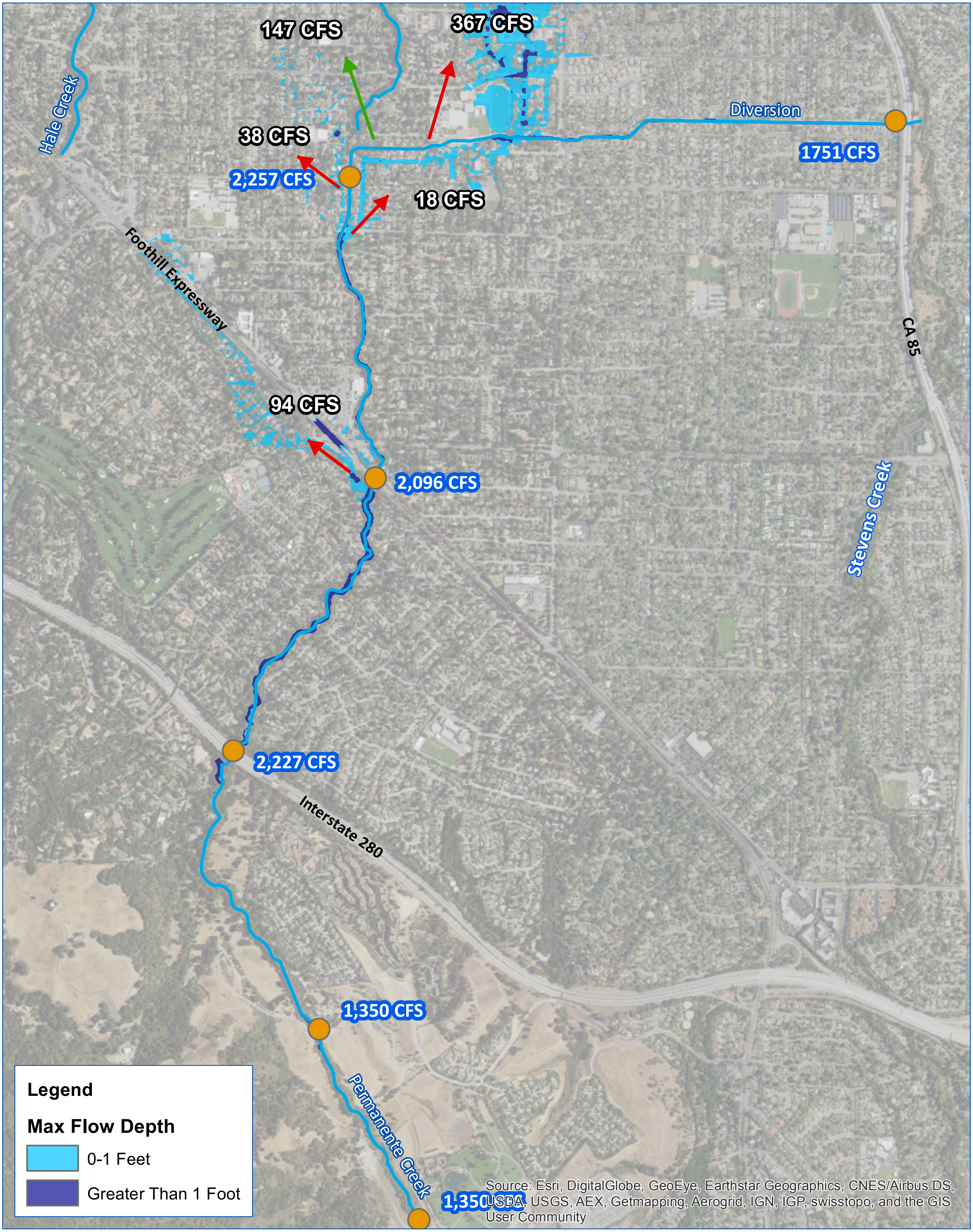
Peak flows where existing channel capacities are exceeded are determined from the steady state HEC-RAS models at Park Avenue and McKelvey Park. These peak flows are then incorporated in the HEC-HMS model as diversions to develop a spill hydrograph for each. The spill hydrographs are then routed in HEC-RAS 5.0, thereby mapping the extents of the shallow flooding. The existing conditions one-percent floodplain and flow schematic is presented as Figure 9 and Figure 10.

Average base flood depths under existing conditions range from less than one foot to ten feet in low lying areas near El Camino Hospital. The predominant depth of flooding is one-half to one foot. Spill from Permanente Creek occurs at Foothill Expressway, Portland Avenue, and between Mountain View Avenue and El Camino Real. The peak spill from the Permanente Diversion Channel near Blach Intermediate School is approximately 280 cfs. The one-percent floodplain extent reaches to U.S. 101.

FLOOD PROTECTION FACILITIES

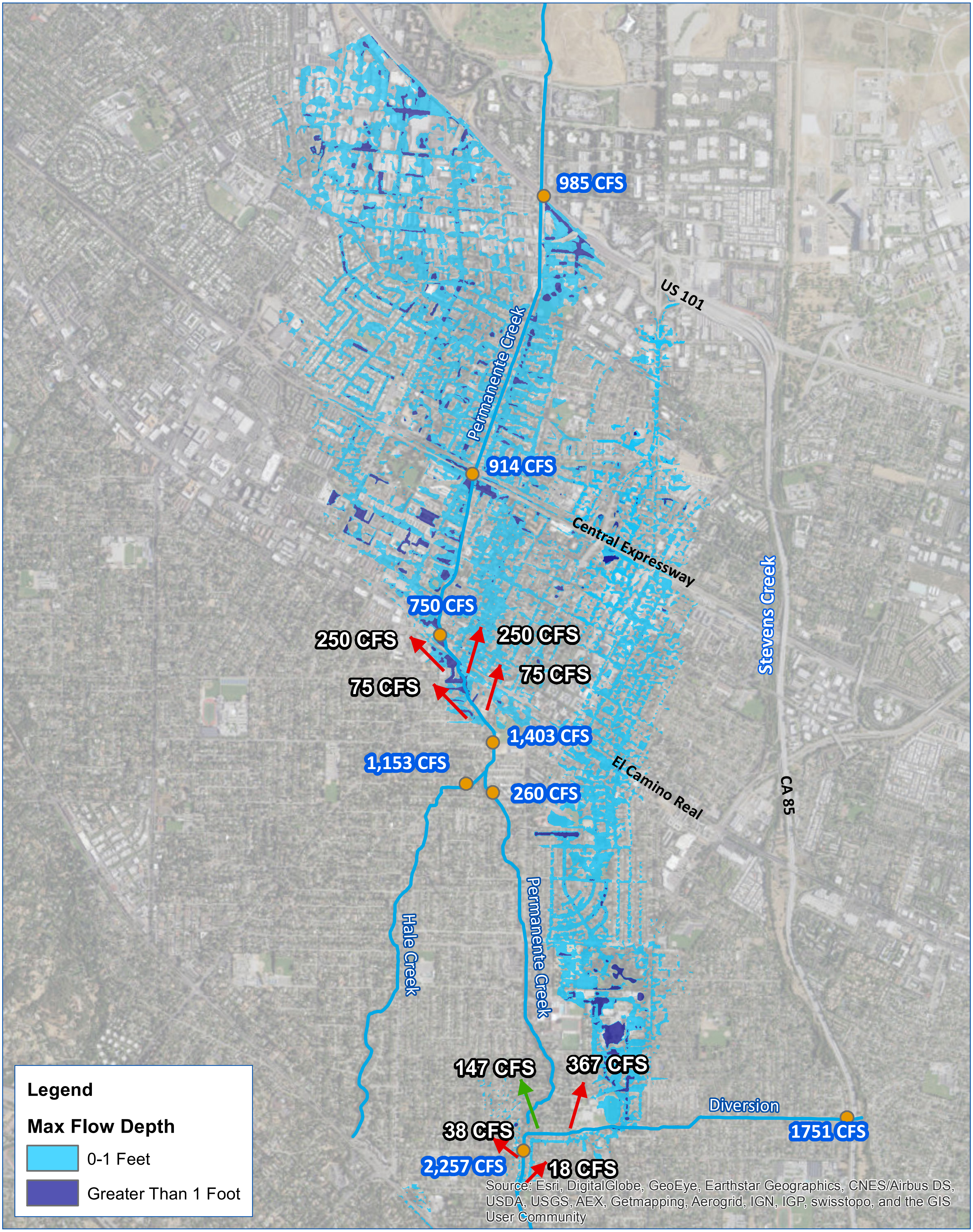
Detailed inflow, storage-elevation and outflow relationships that govern the hydraulic behavior of each diversion or storage facility are defined and flow hydrographs for the base flood event are routed through each basin in its final design configuration using the master HEC-RAS unsteady-state models.

Combined with selective improvements to the Permanente Creek diversion structure and Permanente Creek, the designed flood protection facilities meet the District’s basic objective; that is, the removal of one-percent flooding north of El Camino Real. The design approach is to maximize upstream system storage at Rancho San Antonio Park, given the topographic constraints, a desire to avoid Department of Safety of Dams jurisdiction, and the recreational and aesthetic needs of Santa Clara County Parks.



Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

FIGURE 9: **Existing 1% Flooding Upstream of Diversion**



Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

FIGURE 10: **Existing 1% Flooding Downstream of Diversion**

Flow to Permanente Creek downstream of the Permanente Diversion must be limited to approximately 100 cfs to avoid downstream spills from the capacity-limited Permanente Creek. This results in a spill from the Permanente Diversion Channel at Blach School and a residual floodplain. Two-dimensional flow analysis shows that the residual floodplain does not extend north of El Camino Real, where the storm drain system intercepts the small remaining flow.

Rancho San Antonio County Park Facilities

To provide enough storage capacity to adequately reduce downstream system flows relative to system capacity, the facility at Rancho San Antonio County Park consists of two separate storage basins. A 91 acre-foot basin, referred to as the North Basin, is located north of The Gate of Heaven Cemetery on the east side of Permanente Creek. A smaller 9 acre-foot basin, referred to as the South Basin, is located to the south of the large basin on the west side of the creek. A creek flow diversion structure will be constructed at the Gate of Heaven Cemetery maintenance bridge.

Designed storage-elevation data for the two basins are presented in Table 8, and the corresponding storage-elevation curves are shown in Figure11. It should be emphasized that these are separate basins and while the total amount of available design storage without freeboard is 100 acre-feet, they act independently.

Table 8
Storage-Elevation Data for Rancho San Antonio Basins

Rancho San Antonio South		Rancho San Antonio North	
Elevation (ft)	Storage (ac-ft)	Elevation (ft)	Storage (ac-ft)
436	0.00	394	0.0
437	0.45	395	4.0
438	1.07	396	8.8
439	1.85	397	13.8
440	2.78	398	19.0
441	3.85	399	24.4
442	5.04	400	30.0
443	6.18	401	35.9
444	7.39	402	41.9
445	8.98	403	48.2
446	10.64	404	54.8
		405	61.6
		406	68.6
		407	75.8
		408	83.3
		409	91.0

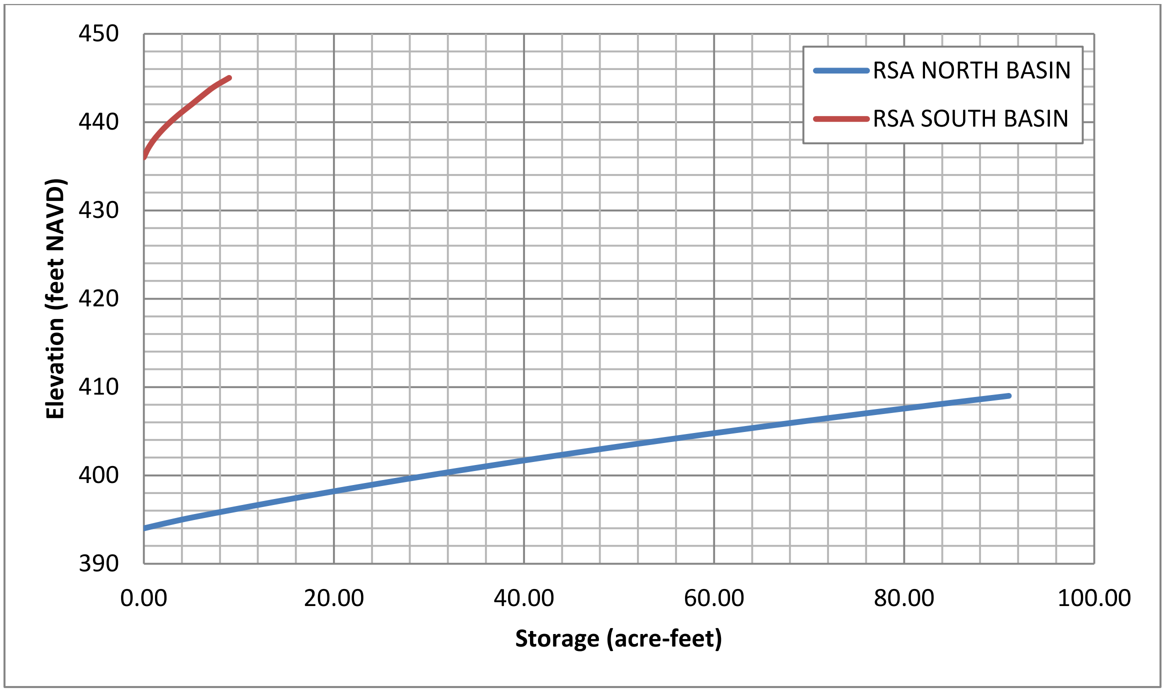


Figure 11: Storage-Elevation Curves for Rancho San Antonio Basins

To force overflows into the two storage basins, a culvert control structure that raises the existing top of the road crossing from an elevation of 437 feet NAVD to an elevation of 445.5 feet NAVD is shown on the final plans. The 12-foot by 6-foot box culvert installed within the crossing serves as a hydraulic choke that limits downstream discharge. The length of the culvert is 50 feet. A flow diversion structure will be constructed at the culvert crossing to direct high creek flows into both storage basins.

The diversion structure consists of a 157-foot long angled box with an overflow weir set at elevation 444.0 feet NAVD. Water that overtops the weir is contained in a concrete structure with is connected to the South Basin by a 36-inch diameter CMP and to the North Bain by an 84-inch diameter CMP. At the diversion box, the South Basin outlet (36-inch) is 438.00 feet NAVD; the North Basin outlet (84-inch) is 434.00 feet NAVD. Rock riprap protection is placed at both outfalls to the storage basins.

Rancho San Antonio Basin Performance

System performance is evaluated with each of the two previously described basins constructed per plan and fully functional. Flood flow routing through each basin is modeled using the unsteady function of HEC-RAS, which is also used to evaluate system performance; that is, whether the limiting flow capacity in Permanente Creek or the Permanente Diversion Channel is exceeded at any downstream location. A final post-project design peak 100-year water surface profile is calculated for the entire system using the HEC-RAS model.

Table 9 summarizes post-project flow routing along Permanente Creek from Rancho San Antonio Park downstream to the confluence with the Permanente Diversion. The total amount of stored water in the system is 75 acre-feet, compared to the 93 acre-feet of storage envisioned for Rancho San Antonio Park by the District in their 2008 Planning Study.

Table 9
Flow Routing from Rancho San Antonio Park to Permanente Diversion

Creek/Reach/Location	Design Capacity (cfs)	Existing 1% Flow (cfs)	Design 1% Discharge (cfs)	Inflow (cfs)	Return Flow (cfs)	Storage (ac-ft)
PERMANENTE CREEK						
Upstream of Rancho San Antonio Park	n/a	1,350	1,350	-	-	-
South Rancho San Antonio Park Basin	-	-	-	79	22	4.4
Rancho San Antonio Park	2,300	1,350	790			
North Rancho San Antonio Park Basin	-	-	-	480	26	70.3
Rancho San Antonio to Interstate 280	2,300	1,350	820	-	-	-
Interstate 280 to Portland Avenue	1,750	2,227	1,727	-	-	-
Portland Avenue to Diversion	2,100	2,257	1,903	-	-	-

Flow is diverted at Rancho San Antonio County Park to reduce peak discharges through the capacity-limited reaches. Figure 12 shows the 100-year flow hydrographs at The Gate of Heaven Cemetery diversion structure, which are summarized by Table 10.

Figure 13 shows the 100-year stage hydrographs for the North Basin and South Basin, respectively. It is noted that the North Basin empties within 65 hours of first spill, and the South Basin empties within 8 hours. This performance is well within Santa Clara County's general requirement that detained water be drained within three to five days for vector control.

Figure 14 demonstrates how design hydrographs are routed from above Rancho San Antonio Park through the storage basins and combined with the runoff from local watersheds at Interstate 280.

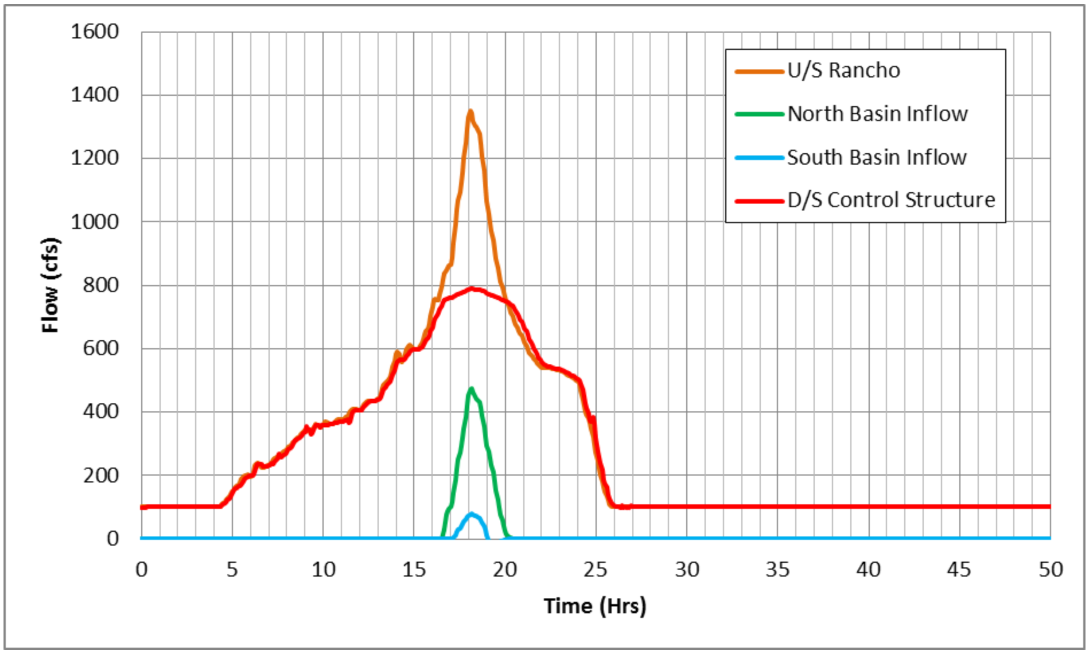


Figure 12: Flow Diversion at Rancho San Antonio County Park

Table 10
Summary of Rancho San Antonio County Park Flood Routing

Routing Parameter	**Value**
CONTROL STRUCTURE	
Peak Upstream Discharge on Permanente Creek (cfs)	1,350
Peak Discharge through Control Structure (cfs)	560
Peak Discharge to North Storage Basin (cfs)	481
Peak Discharge to South Storage Basin (cfs)	79
Peak Recombined Flow Downstream on Permanente Creek (cfs)	820
NORTH STORAGE BASIN	
Inflow Volume (ac-ft)	76
Detained Volume (ac-ft)	70.3
Maximum Elevation in Basin (feet NAVD)	406.3
Peak Return Flow (cfs)	26
Time to Drain (hours)	65
SOUTH STORAGE BASIN	
Inflow Volume (ac-ft)	7.1
Detained Volume (ac-ft)	4.4
Maximum Elevation in Basin (feet NAVD)	441.5
Peak Return Flow (cfs)	22
Time to Drain (hours)	8

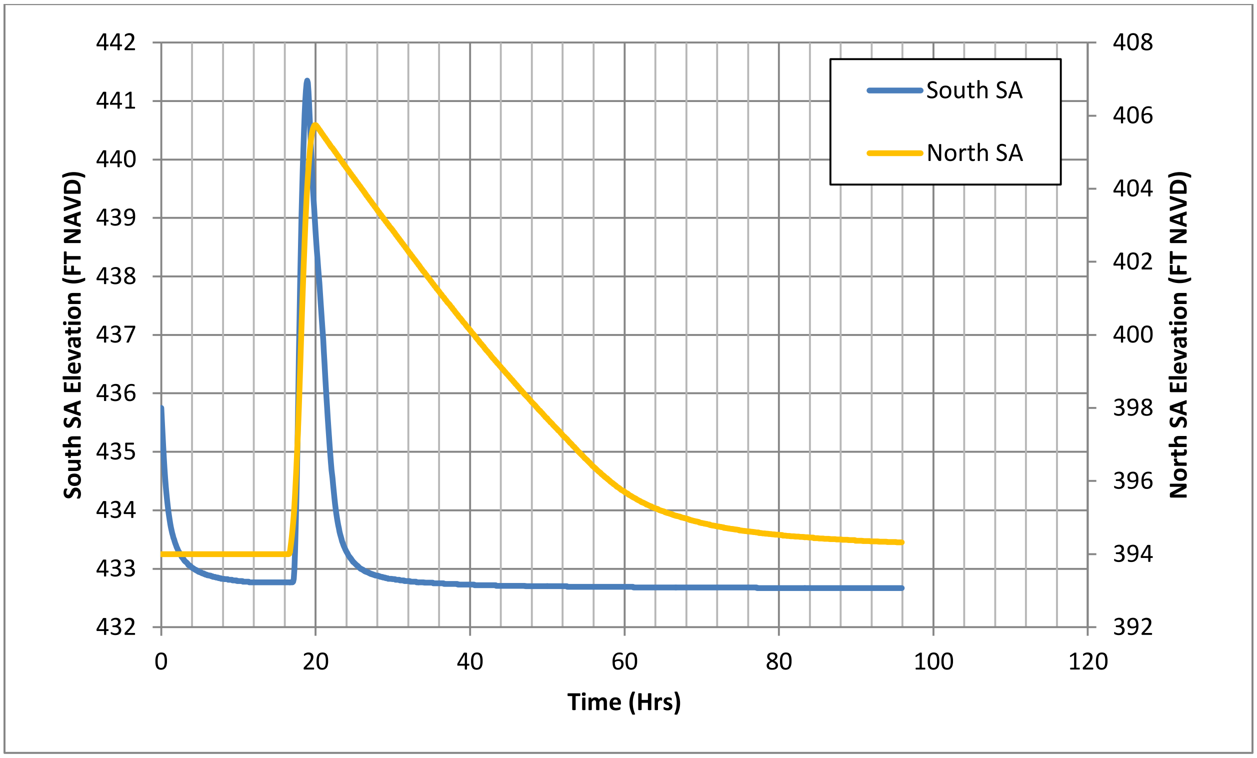


Figure 13: Stage Hydrographs at Rancho San Antonio County Park Basins

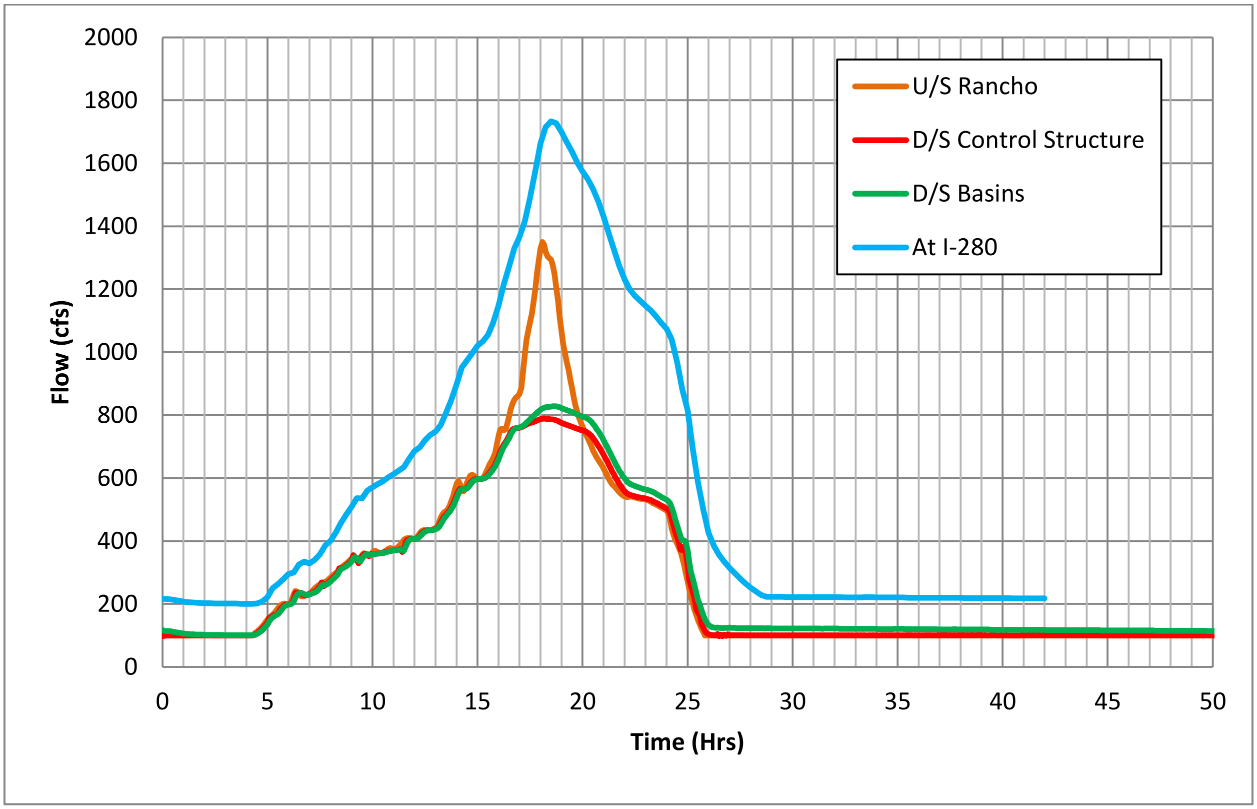


Figure 14: Permanente Creek Hydrograph Routing to Interstate 280

Permanente Creek Diversion

Based on HEC-RAS flood hydrograph routing, the peak discharge at the confluence of Permanente Creek with the Permanente Diversion is 1,903 cfs after the completion of the Rancho San Antonio County Park storage facilities. Hydraulic analysis using the unsteady state flow model indicates that the capacity of the Diversion channel between Permanente Creek and Stevens Creek is roughly 1,700 cfs. Additional hydraulic analysis concludes that the limiting capacity of Permanente Creek between the Diversion and the confluence with Hale Creek is about 200 cfs due to bottlenecks located between Cuesta Drive and Marilyn Drive. Furthermore local runoff enters Permanente Creek within this capacity constricted reach. Thus, regardless of how the discharge is split between Permanente Creek and the Diversion, the total flow is in excess of total system capacity. Therefore flow must be removed from the system.

As shown photographically in Figure 15, the Permanente Diversion consists of a 7-foot wide by 1-foot high opening in the floor of the Permanente Diversion Channel that leads to the box structure, with its elevation shown in Figure16.

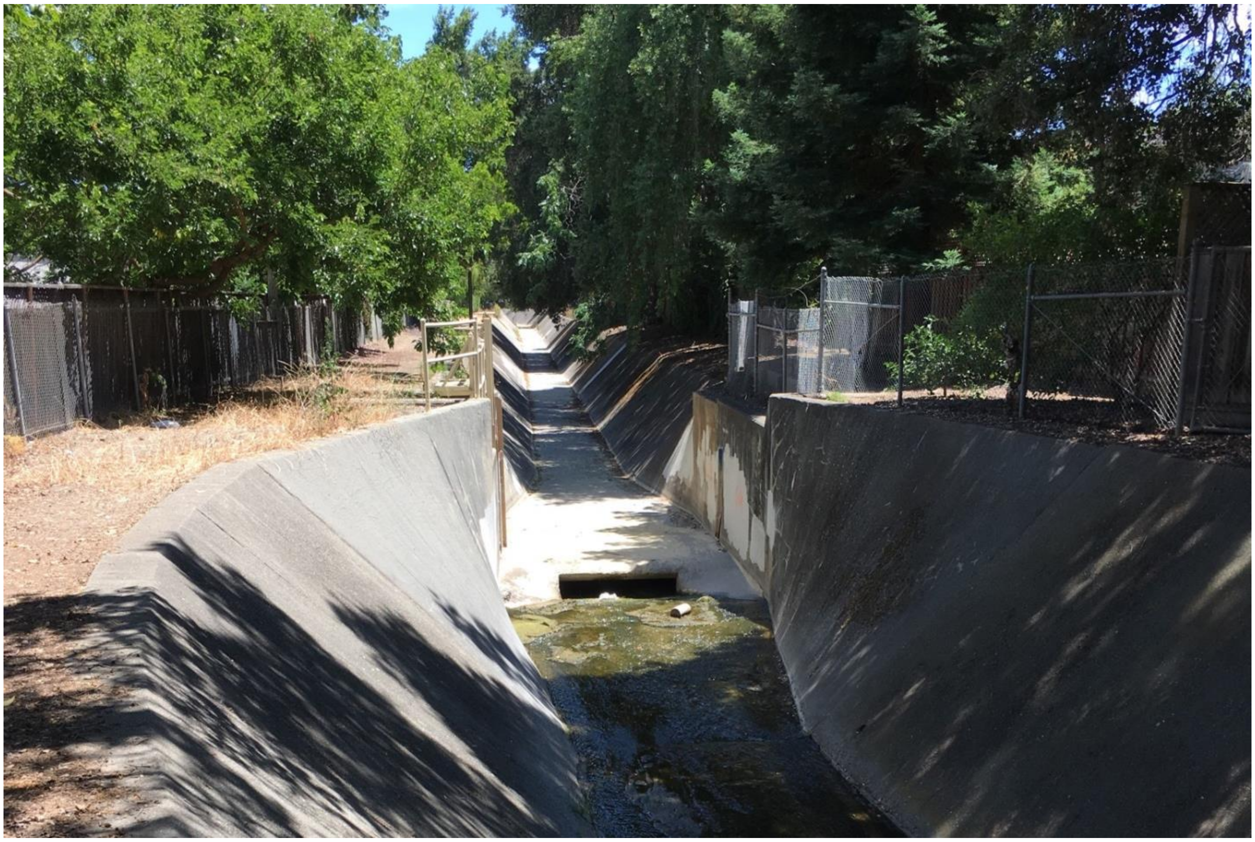


Figure 15: Permanente Creek Low Flow Diversion

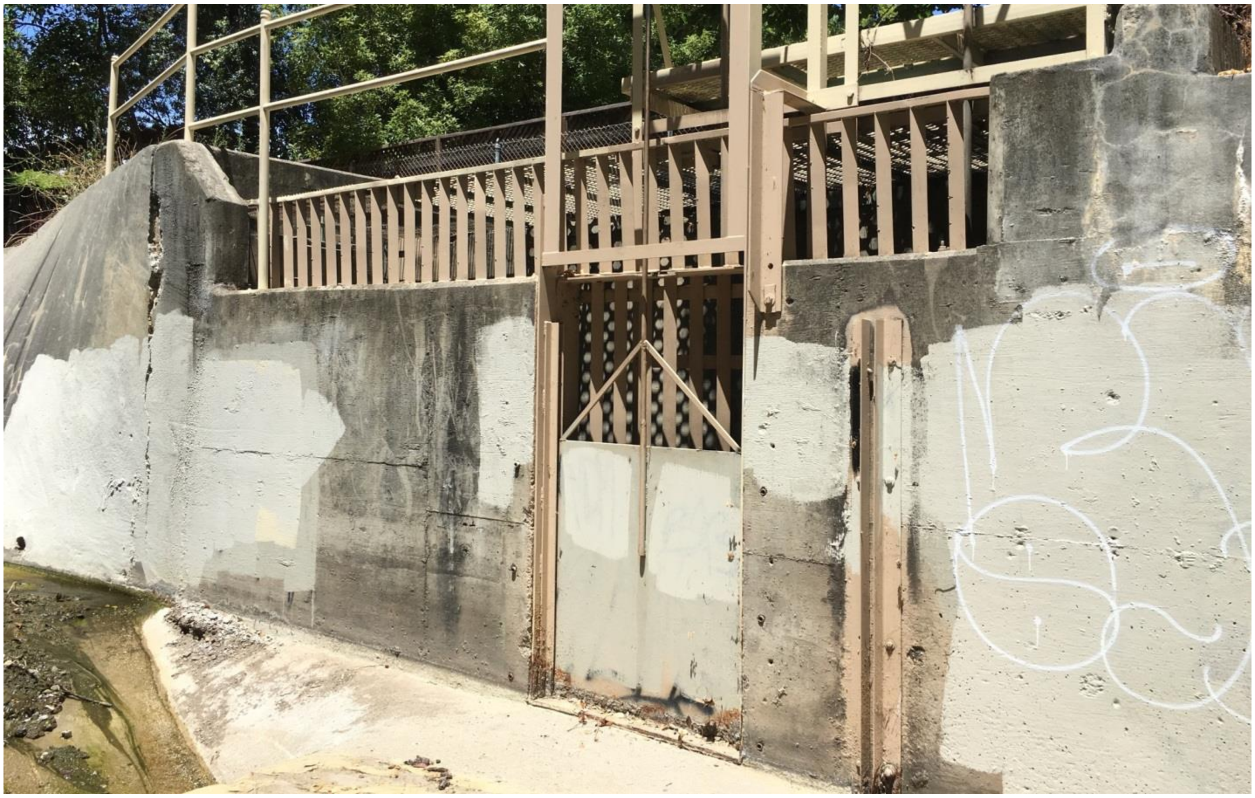


Figure 16: Permanente Creek Diversion Weir and Gate

This structure is drained by a 54-inch diameter RCP that crosses under Eastwood Drive and discharges to the open Permanente Creek. The diversion structure includes a 17 feet long weir at elevation 187.7 feet NAVD (6.9 feet above the channel floor). There is also a full-height opening in the wall 3.5 feet in width with an operable slide gate that can close off about 56 percent of the height of the opening.

With the low flow diversion and weir configured as shown in Figures 15 and 16, nearly 150 cfs discharges to the original Permanente Creek alignment if the low flow inlet remains unobstructed. (This flowrate is valid for both the existing condition and post-project condition, as the diversion rate is not affected by backwater conditions on Permanente Creek.)

While the downstream channel has sufficient capacity to deliver 150 cfs to the proposed McKelvey Park diversion weir, the McKelvey Park basin would overfill with this discharge rate at the Permanente Diversion. Consequently, the rate of flow to Permanente Creek must be limited to roughly 100 cfs as was the original District intent for the McKelvey Park facilities to work as planned.

Necessary Operational Changes to Diversion

Unsteady hydraulic modeling demonstrates that this flow limitation can be achieved by closing the 3.5 feet wide gated opening completely and eliminating the overflow weir by pouring concrete to an elevation of 190.7 feet NAVD to match the top of existing channel wall. (The calculated one-percent post-project water surface elevation at this location is 190.0 feet NAVD.)

The overflow was likely intended to provide a "safety valve" in the event the diversion inlet on the bottom of the channel becomes plugged with debris and/or sediment, which has apparently happened in the past. (With a plugged low flow inlet, the weir as presently configured could divert about 270 cfs to the original Permanente Creek alignment during a one-percent event.) Given the flow capacity restrictions between the Permanente Diversion Channel and the Hale Creek Confluence, however, it is actually safer to cap the maximum rate of diverted flow to the original Permanente Creek alignment. Should the low flow inlet become plugged additional flow will discharge down the Permanente Diversion Channel toward Stevens Creek and spill into the Blach Intermediate School. The overflow floodplain would expand and deepen only slightly due to the additional 100 cfs out of 1803 cfs (an increase of only 6 percent). Otherwise there could be induced flooding to the north from unanticipated spill out of Permanente Creek and the McKelvey Park storage basin.

It is also advisable to leave a small block-out opening in the closure wall just above the invert of the diversion channel, outfitted with a new slide gate. This would help crews maintain the low flow inlet as the diversion channel could be temporarily blocked (e.g. with sandbags) with the gate is opened to divert water into the natural Permanente Creek channel. A 3.5-foot by 2-foot opening would limit the maximum flow to the creek under one-percent discharge conditions to less than 100 cfs in the event of complete low flow inlet blockage.

Performance of Reconfigured Permanente Creek Diversion

With diverted flow limited to 100 cfs, hydraulic analysis indicates there would be a spill out of the Permanente Diversion of roughly 115 cfs near Blach Intermediate School, noting this spill is less than the existing conditions one-percent spill at the same location of approximately 367 cfs. Figure 17 shows hydraulic conditions at the Permanente Diversion spill location, including the upstream discharge, spill to Blach Intermediate School and discharge to Stevens Creek, for both existing and post-Project conditions. The Project, specifically the creation of diversion storage at Rancho San Antonio County Park and the limitation of flow to the downstream reach of Permanente Creek, does not increase discharge to Stevens Creek.

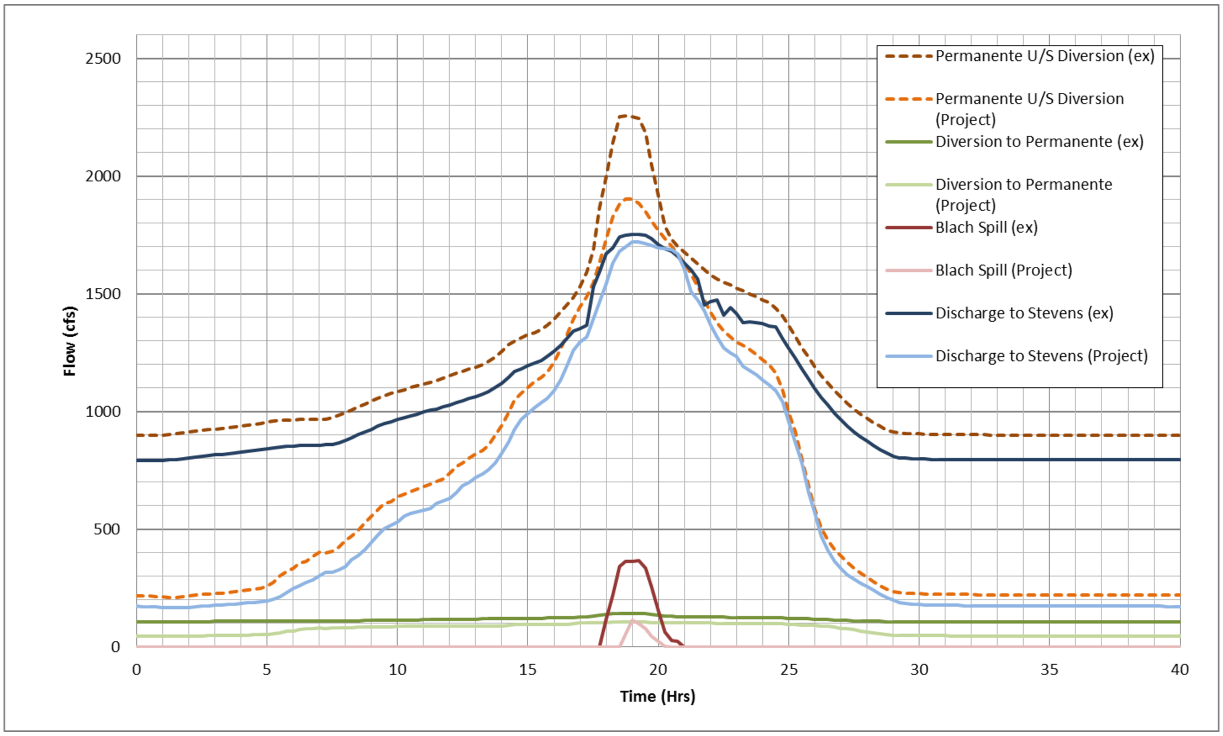


Figure 17: Permanente Diversion Channel Spill Hydrographs

McKelvey Park Facilities

The proposed McKelvey Park storage basin receives flood flows that exceed the downstream capacity of Permanente Creek just below its confluence with Hale Creek, assuming that a future project to rehabilitate Hale Creek enables the peak Hale Creek discharge to be delivered to this location. Storage-elevation data for the McKelvey Park improvement design are presented in Table 11, and the corresponding storage-elevation curves are shown in Figure 18.

Table 11 Storage-Elevation Data for McKelvey Park Basin

McKelvey Park	
Elevation (ft)	**Storage (ac-ft)**
90.0	0.0
104.5	56.7

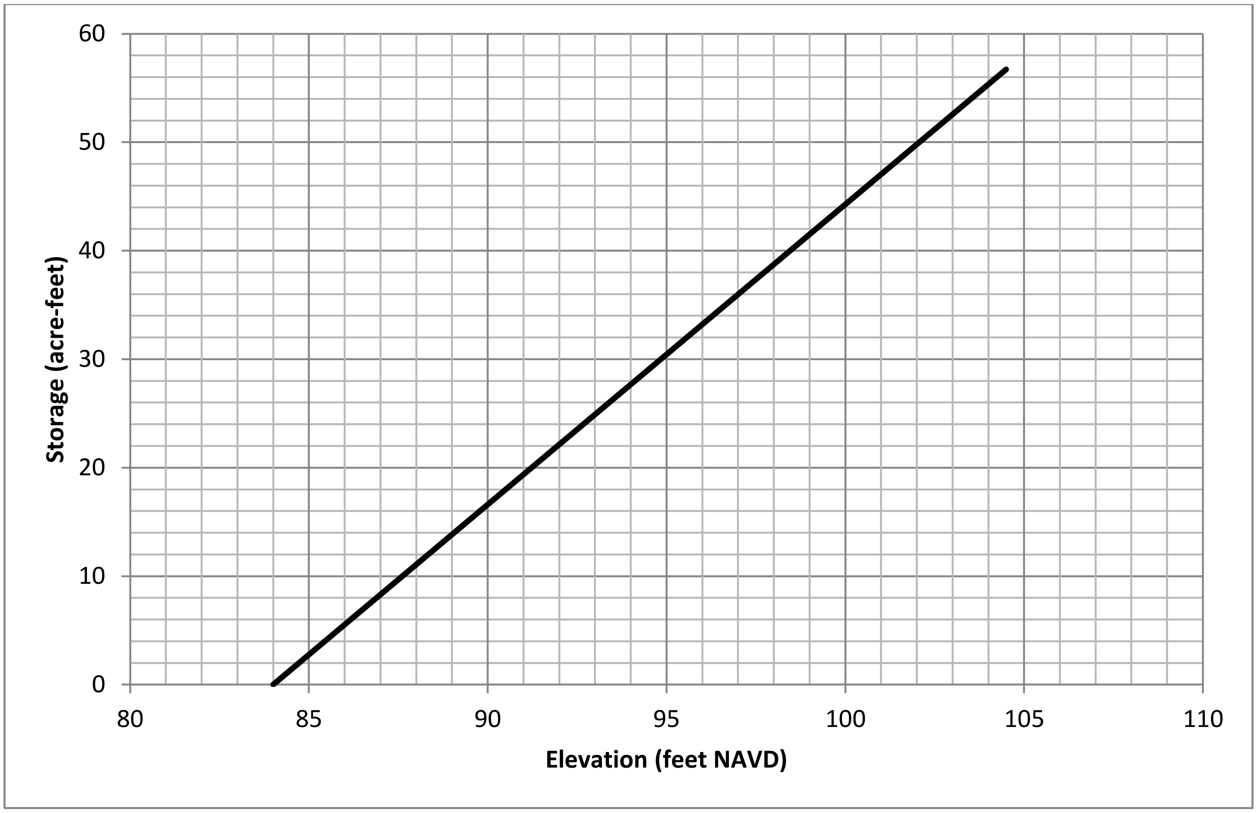


Figure 18: Storage-Elevation Curve for McKelvey Park Basin

Excess flow is diverted to the park via a 207 feet long lateral weir at elevation 108.00 feet NAVD, located on the east side of Permanente Creek just upstream of the Mountain View Avenue Bridge. The bridge acts as a hydraulic control, limiting downstream flows and forcing overflow to the new McKelvey Park basin. Hydraulic performance is fairly sensitive to the size of the Mountain View Avenue Bridge opening, but the existing culvert entrance (12 feet wide by 7 feet high) does not require modification.

Water flowing over the weir will fall into an 18 feet wide truck ramp that directs the overflow into McKelvey Park. A pedestrian bridge crosses the excavated open control structure, at the original park grade. Two Flygt NP 3171 LT (Impeller 614) pumps will discharge stored floodwaters back into Permanente Creek at the south end of the overflow weir structure once the storm has passed. The pumps will also serve to dewater the basin for any local park drainage.

McKelvey Park Basin Performance

Table 12 and Figure 19 summarize the hydraulic performance of the new McKelvey Park storage basin, based on post-Project conditions in the one-percent design storm. The new basin will store 50.7 acre-feet of storm water to a maximum elevation of 102.9 feet NAVD.

The McKelvey Park basin would empty within 53 hours of the first overflow from Permanente Creek. This performance is well within Santa Clara County's general requirement that detained water be drained within three to five days for vector control. Figure shows the resulting one-percent floodplain after the construction of storage at Rancho San Antonio County Park, a reconfigured Permanente Diversion, and the completion of McKelvey Park facilities.

Table 12
Summary of McKelvey Park Flood Routing

Routing Parameter	**Upstream Diversion**
CONTROL STRUCTURE at Mountain View Avenue	
Peak Upstream Discharge in Permanente Creek (cfs)	1,366
Peak Discharge thru Mountain View Ave Bridge (cfs)	1,012
Peak Discharge to McKelvey Park Basin (cfs)	365
STORAGE BASIN	
Inflow Volume (ac-ft)	56.0
Detained Volume (ac-ft)	50.7
Maximum Elevation in Basin (feet NAVD)	102.9
Peak Pump Station Flow (cfs)	15
Time to Drain (hours)	53

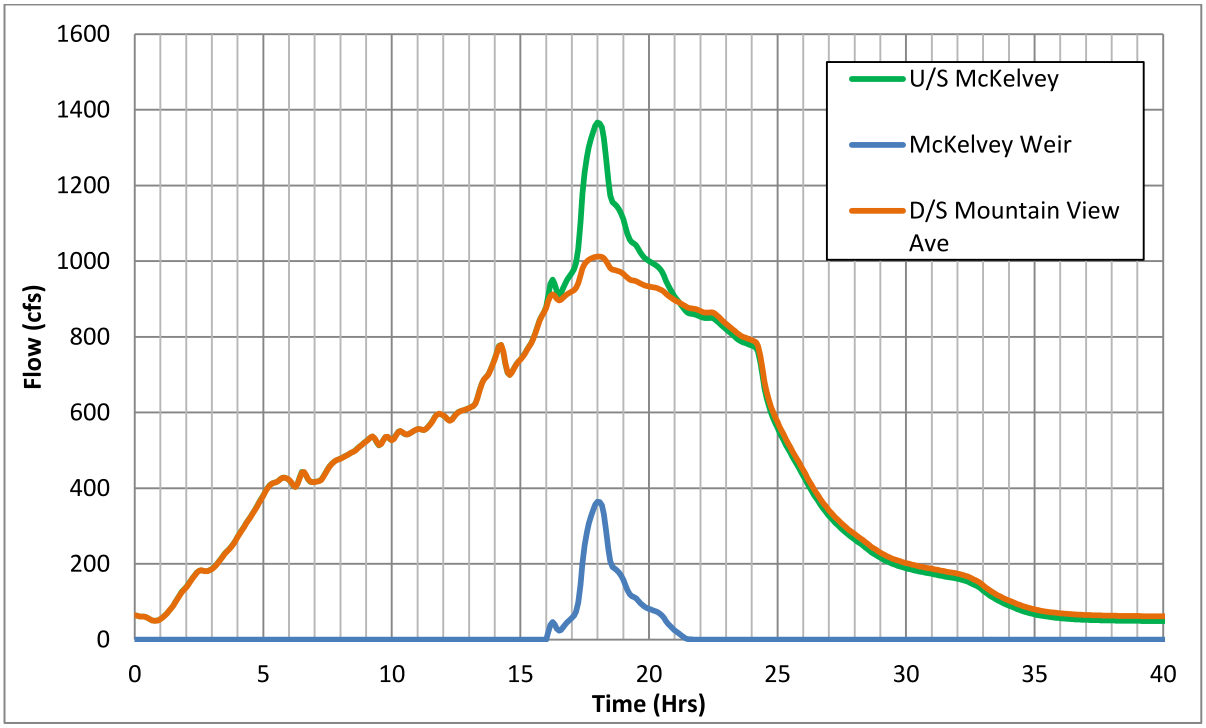
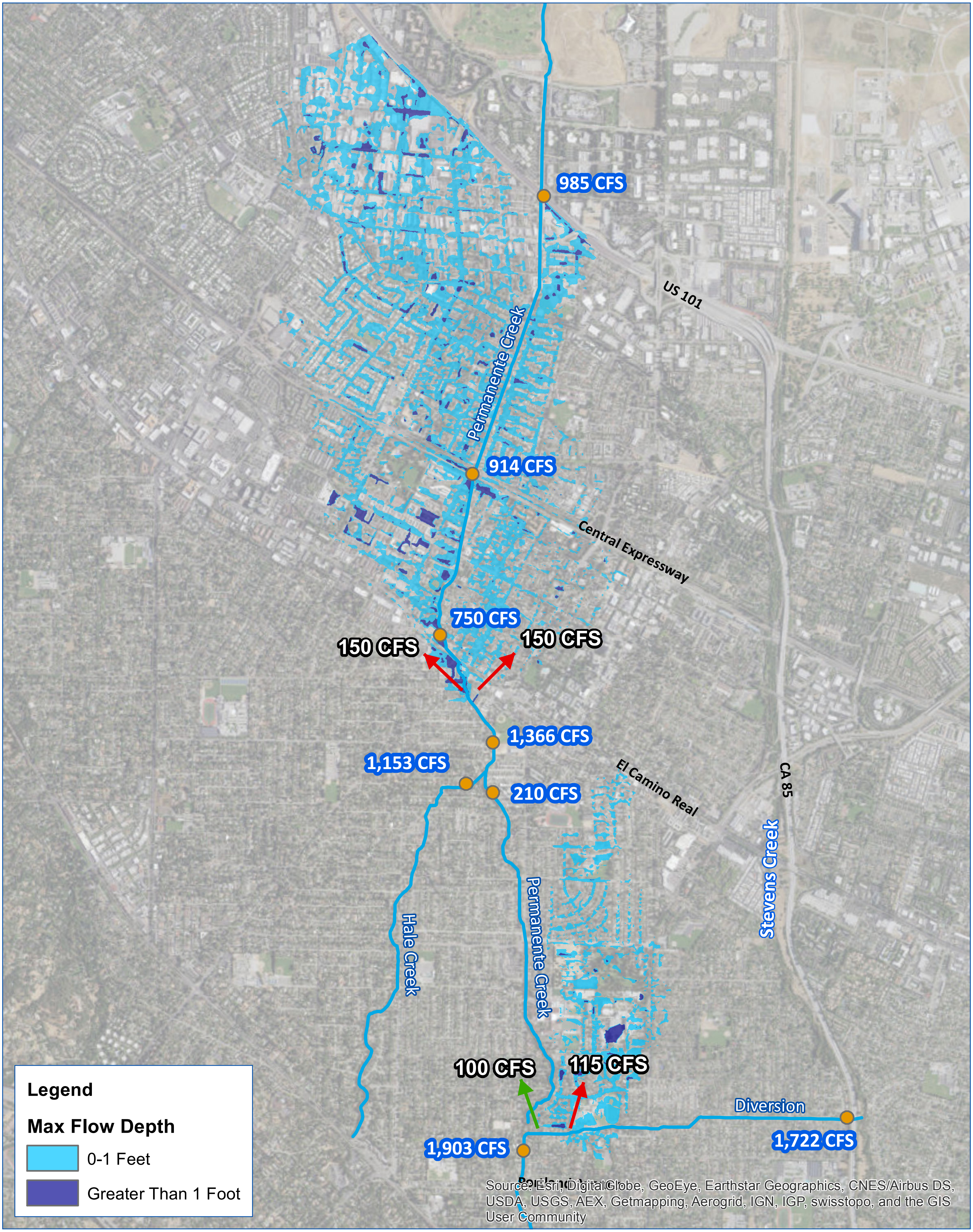


Figure 19: Routing through McKelvey Park Basin



Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

FIGURE 20: **Extent of 1% Flooding w/ McKelvey Park Basin**

Storage in the McKelvey Park basin attenuates the peak post-Project one-percent discharge in Permanente Creek from 1,366 cfs to 1,012 cfs at El Camino Real. Unfortunately the natural creek bank elevations in the vicinity of Park Drive are not sufficiently elevated to contain all of this discharge and 300 cfs spills to the north and west as shown in Figure 20. This overflow is generally less than one foot in depth and would be mapped as Zone X, but the Project objective of protecting areas north of El Camino Real from the base flood may not be considered as completely met.

This spill can be remediated by constructing floodwalls on both sides of Permanente Creek for 400 feet upstream of Park Drive. The maximum height of floodwall to contain the one-percent water surface elevation is 2.4 feet. Some sort of nominal freeboard (e.g. one foot) is recommended; however, providing full freeboard to FEMA standards is not recommended because:

1. Maximum wall height would be 6.5 feet.
2. Assuming non-accreditation and floodwall failure for mapping purposes, the affected area would remain Zone X with or without full freeboard to FEMA standards.
3. In terms of relative risk, a minimum one foot of freeboard plus or minus is afforded to the one-percent water surface profile in adjacent upstream and downstream reaches of Permanente Creek. There is no reason to provide additional flood risk mitigation within this limited reach of the creek.

With the entire post-Project one-percent discharge contained at Park Drive, the downstream channel has sufficient capacity to deliver its peak one-percent discharge to San Francisco Bay without further spill.

PROJECT PERFORMANCE SUMMARY

With all elements of flood protection constructed as described herein, the Project design meets the stated Project objective, which is to remove the special flood hazard zone north of El Camino Real. Figure 21 schematically details post-Project base flood routing for design peak discharges.

Post-Project Floodplain

Figure 22 provides the post-Project one-percent floodplain and a flow schematic. This figure illustrates hydraulic model results that document the following:

1. The Project meets its design objective, with the elimination of the one-percent riverine floodplain north of El Camino Real; and

2. The Project does not cause an expansion in the one-percent floodplain, either in depth or extent.

Project Benefit

The main Project benefit is to eliminate or reduce these mapped flood hazard zones. All else equal, the elimination or reduction of flood hazards, particularly those presently shown on the effective Flood Insurance Rate Map, enhance property values.

Ten-Year Flood Event

Public outreach programs that have supported the District’s 2008 Planning Study have generally indicated that overflows into the various off-stream storage basins would not occur during events more frequent than the ten-year return period. (Table 1 provides a synopsis of the probability of experiencing a ten-year event over various periods of time.) Flows from Table 2 are used to assess post-Project performance during a design ten-year event following the same flood routing methodology. Hydraulic modeling shows that all ten-year peak discharges are safely conveyed in the Permanente Creek flood control system as designed, and there would be no anticipated spill into either the Rancho San Antonio County Park storage basins or the McKelvey Park storage basin.

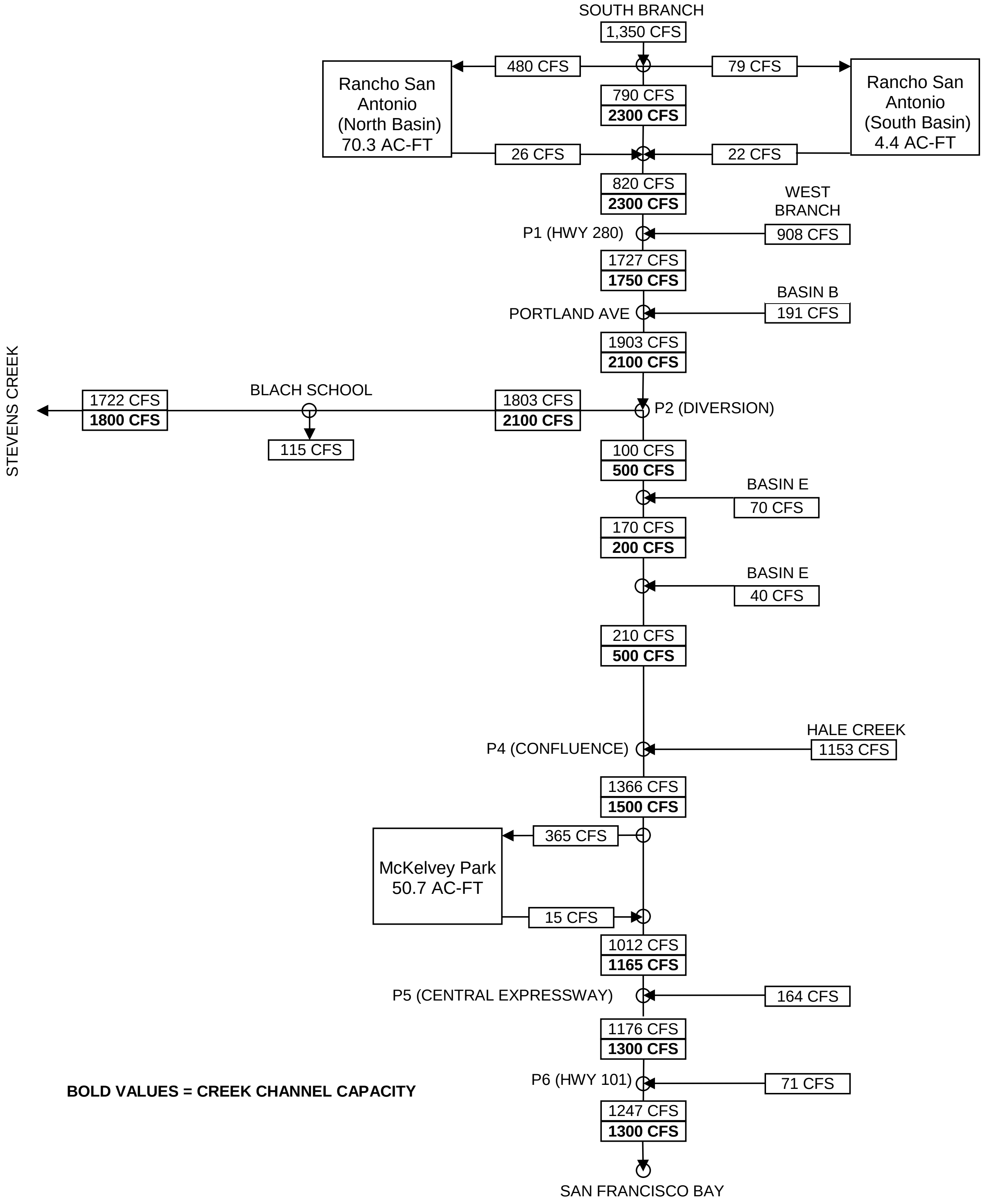
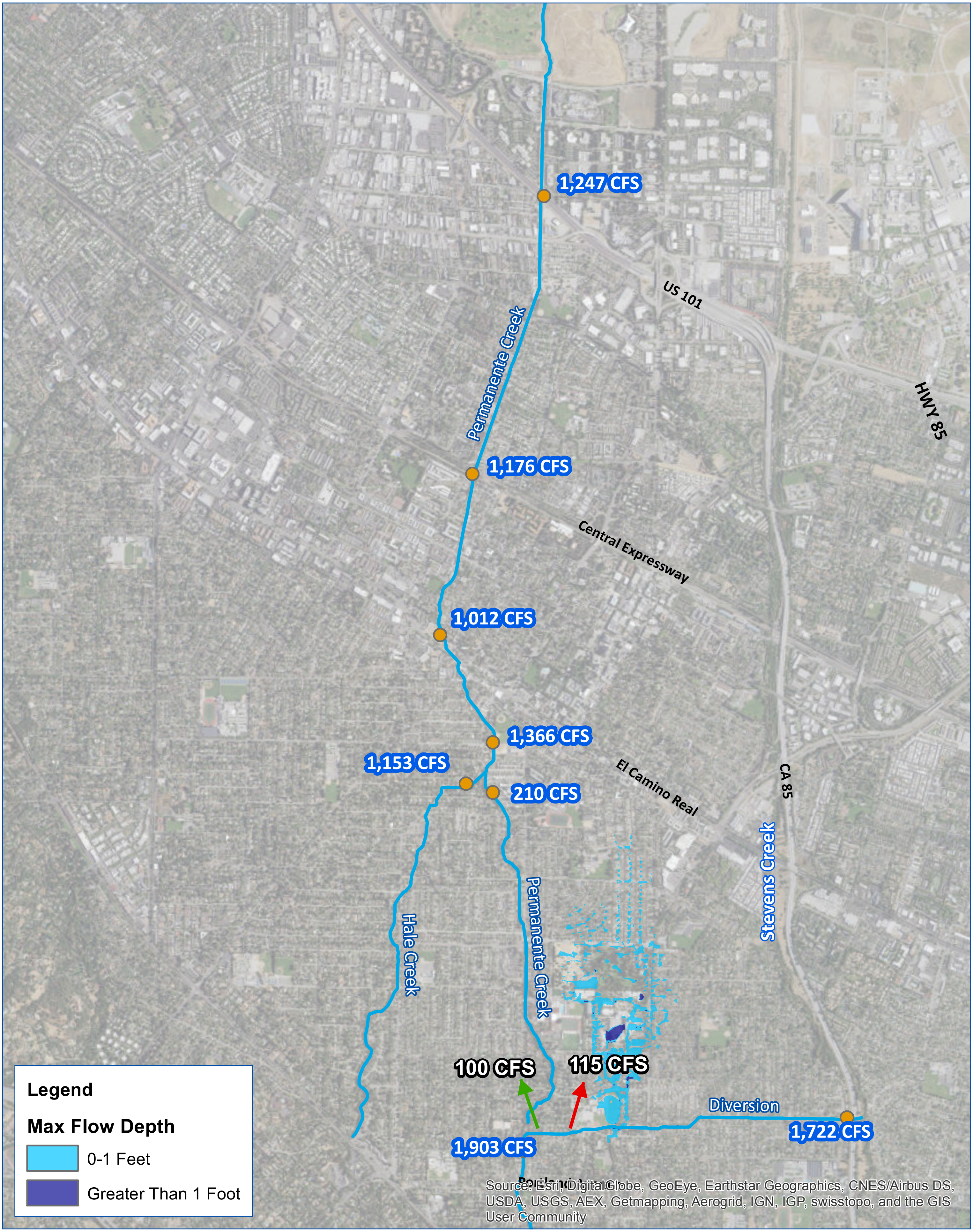


Figure 21: Base Flood Routing Schematic (Post-Project)



Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

FIGURE 22: **Proposed 1% Flooding**

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